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THE MECHANISMS OF SWELLING IN
CLAY SOILS

by

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A THESIS

SUBMITTED TO THE FACULTY OF GRADUATE STUDIES
IN PARTIAL FULFILMENT OF THE REQUIREMENTS FOR THE DEGREE
MASTER OF SCIENCE

DEPARTMENT OF CIVIL ENGINEERING

EDMONTON, ALBERTA

APRIL, 1965

UNIVERSITY OF ALBERTA

FACULTY OF GRADUATE STUDIES

The undersigned certify that they have read,
and recommend to the Faculty of Graduate Studies for
acceptance a thesis entitled "THE MECHANISMS OF SWELLING
IN CLAY SOILS", submitted by PHYZOOOL ALI in partial
fulfilment of the requirements for the degree of Master
of Science.

ABSTRACT

The problem on which this thesis was based was the investigation of the mechanisms of swelling in a highly plastic clay soil. The basic cause of swelling of a clay mass was investigated by considering the various postulates set forth in the literature to explain the mechanisms of swelling. For engineering purposes it is convenient to study swelling behaviour at various moisture contents.

The particular clay soil used was Lake Edmonton clay treated to produce sodium and calcium modifications. The samples were prepared at initial moisture contents approximating the liquid limit and then desiccated to moisture contents of approximately 50%, 30%, 22%, 16% and 1%. The no-volume-change pressure was measured in the one dimensional consolidometer using immersing solutions of various concentrations. In addition one test on the clay was performed using varsol as the immersing solution and two tests were run on pulverized sand samples.

It is postulated that osmosis between the adsorbed water and the pore solution is the major contributor to the soil swelling for initial moisture contents ranging from slightly below to considerably above the plastic limit. In this range of moisture contents the capillary

effect, elastic restitution of the soil particles and osmosis between the soil mass and external solution were found to contribute to swelling to a minor degree. Below the shrinkage limit particle hydration would induce swelling pressures only if the clay were in a saturated state.

During the testing program it appeared that the rate of loading and degree of consolidation influenced the swelling behaviour of the soil. It is therefore recommended that a study of stress history on swelling behaviour be considered as a logical continuation of this work.

ACKNOWLEDGMENTS

The author extends sincere appreciation to the following concerning the work for this thesis:

Dr. S. Thomson, Associate Professor, Department of Civil Engineering, for his ever helpful guidance throughout the course of the study.

Dr. S. Pawluk of the Department of Soil Science for giving freely of his time whenever needed to discuss special problems.

The staff of the Alberta Soil Survey Laboratory, the Department of Mining and Metallurgy, and the staff of the Machine Shop for their assistance from time to time.

The Department of Civil Engineering and the Alberta Research Council for providing financial assistance.

To my wife, Myrtle, for typing the manuscript and for her encouragement.

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GLOSSARY OF TERMS AND SYMBOLS

TERMS

Adsorbed Water: Water in a soil mass, held by physico-chemical forces, having physical properties substantially different from ordinary water, at the same temperature and pressure.

Cation Exchange Capacity: The quantity of cations that can be adsorbed and exchanged by a clay. It is considered to be a characteristic constant for a soil and is measured in milligram-equivalents or milliequivalents of cation per 100 grams of dry soil.

Desiccation: The phenomenon of moisture loss from a soil due to atmospheric conditions.

Evapo-transpiration: The combined effect of evaporation and transpiration.

Homionic Soil: A soil with only one type of cation adsorbed on its surfaces.

me/100 gms. a.d.s.: Milliequivalents per 100 grams of air dried soil. A milliequivalent is the amount of a reagent required to combine and react with 1/1000th of an atomic weight of hydrogen.

Potential: Refers to electric potential. The equilibrium potential state refers to the condition when the electric potential of the clay micelle is in equilibrium with the external conditions.

Total Exchange Capacity: The total quantity of cations in me/100 gms. a.d.s. as determined by the flame photometer and includes both those cations adsorbed and those present in the pore water.

SYMBOLS

A	:	Area
β	:	Compressibility of water
C	:	Concentration in moles/litre
Ca Ac ₂	:	Calcium Acetate
Cc	:	Compressive Index
Σ E.C.	:	Sum of Exchange Capacity
C _m	:	Concentration of Ions in the Mid-Plane Between Particles
C _o	:	Concentration of Ions in the External Solution
d	:	Distance
e _i	:	Initial Void Ratio
e _f	:	Final Void Ratio
f	:	Activity Coefficient
H _i	:	Initial Height of Sample
HOH	:	Water
i	:	Grams of Water per 100 grams Dry Soil
L.E.C.	:	Natural Remoulded Lake Edmonton Clay
ω_L	:	Liquid Limit
M	:	Molar (concentration)
N	:	Normality (concentration)
Na Ac	:	Sodium Acetate
N.P.	:	Non plastic

P	:	Pressure Kg/cm ²
I _p	:	Plasticity Index
P(ideal):	:	Ideal Osmotic Pressure Kg/cm ²
ω _p	:	Plastic Limit
P _{max}	:	Maximum Pressure Kg/cm ²
P _{min}	:	Minimum Pressure Kg/cm ²
P _s	:	Osmotic Pressure or Swelling Pressure Kg/cm ²
Δ P	:	Pressure Change Kg/cm ²
R	:	Universal Gas Constant
G _s	:	Specific Gravity of Soil Solids
S _{ri}	:	Initial Degree of Saturation
C _s	:	Swelling Index
ω _s	:	Shrinkage Limit
Soln.	:	Solution
T	:	Absolute Temperature °C
"T"	:	Exchange Capacity
T.E.C.	:	Total Exchange Capacity
V _a	:	Valence
% Δ V	:	Percent Volume Change
V _w	:	Volume of Water
Δ V _w	:	Change in Volume of Water
ω _i	:	Initial Moisture Content (Percent)
ω _f	:	Final Moisture Content (Percent)
ψ	:	Potential
W _s	:	Weight of soil solids in grams

CHAPTER I

INTRODUCTION

1.1 General

During recent years engineers have been confronted with problems in swelling soils in which current theories have been found inadequate in the prediction of actual behaviour. In addition to unusually low strengths observed, failures have occurred from excessive heaving. The problem of swelling soils is of specific interest in Western Canada because most of the soils in the area exhibit excessive expansive characteristics. The normally consolidated glacial lake deposits in Manitoba, Saskatchewan and Alberta, as well as the overconsolidated Bearpaw formations in this area show the same high swelling characteristics.

The phenomenon of swelling in soils has long been recognized by both soil scientists and engineers. The soil scientists were the first to embark in this field of study while the soils engineer coped with the problem as best as he could from intuition, until Terzaghi introduced the theory of consolidation and postulated that swelling was the opposite of consolidation. Studies during the past thirty years on

swelling soils have yielded several explanations for the swelling behaviour. These postulates are based on mechanical, physical, and physico-chemical concepts and include the capillarity effect, particle hydration, cation hydration, osmotic action, and purely elastic rebound. These concepts will be discussed in more detail in subsequent chapters. The research of this thesis was to attempt an investigation of these postulates to assess the role, if any, each plays in the swelling of a remoulded sample of Lake Edmonton clay.

1.2 Scope of the Investigation

The investigation was concerned with at least a qualitative determination of the mechanisms that cause swelling in a clay. This was attempted by evaluating the existing theories and by completing a comprehensive testing program.

The testing program consisted of constant volume consolidation tests. The soil was a highly plastic clay, treated to yield homionic modifications with sodium and calcium as the adsorbed cations. These cations were chosen because they are of common occurrence in local natural clay soils and it was known from previous work (Locker, 1963, Thomson, 1963), that their effects

represent general extremes of soil behaviour of interest to the engineer. The samples were prepared at approximately the liquid limit and were desiccated or oven dried to initial moisture contents of about 50%, 30%, 22%, 16% and 1%. For each of these moisture contents a test series was conducted by varying the immersing solution. One test on the Lake Edmonton clay was performed using varsol as the immersing solution. In addition, tests were performed on samples of pulverized Ottawa sand which was considered chemically inert in comparison with the clay.

1.3 Organization of the Thesis

A brief literature review pertinent to this thesis is presented in Chapter II. Chapter III is concerned with sample preparation and the testing method. This is followed by a presentation and discussion of the test results in Chapter IV. Chapter V is devoted to a discussion of the mechanisms of swelling arising from an analysis of the test results and from the literature review. Conclusions from this investigation are presented in Chapter VI and recommendations for future research, in Chapter VII.

1.4 Summary

In summary, the general problem concerned the

determination of the causes of swelling in a clay. Specifically the problem was to attempt an investigation of the various postulates pertaining to the swelling in order to assess the role, if any, each plays in the swelling of a remoulded sample of Lake Edmonton clay.

CHAPTER II

LITERATURE REVIEW

2.1 Introduction

There exist many papers that deal with swelling soils, some of which outline practical problems while others are based on research into the swelling phenomena. This chapter reviews typical problems of swelling and then the theoretical explanations.

2.2 Typical Problems

Engineering problems of swelling soils may be considered to occupy three groups viz.:

- (a) Swelling due to changes in the moisture equilibrium of the soil.
- (b) Swelling that results from a removal of previous loading.
- (c) Swelling due to changes in the ambient solution in the pores of the soil.

For any particular case the amount of total swelling could result from one or all of these causes.

Consider a soil in which the moisture condition is normally affected by evapo-transpiration. Once a structure is built on this soil, the effect of

evapo-transpiration may be reduced. The moisture accumulates in the soil below the structure resulting in a moisture content increase to a point beyond that achieved in its past history. This increase in moisture causes the soil to swell and may result in failure, (Collins, 1957)*. However, an increase in soil moisture condition can also result from irrigation projects, increase in piezometric levels due to damming for a reservoir, or any other project that will cause the ground water level to assume a higher position and cause detrimental swelling.

For a soil subjected to removal of overburden, an immediate expansion occurs upon unloading. In nature, this soil may eventually come in contact with water and further swelling results. The amount and rate of swelling depends on the type and degree of previous loading. Besides the increase in moisture content, the soil could be subjected to leaching with a less concentrated solution (surface water) which could substantially increase swelling. Such conditions exist in the overconsolidated formations of Western Canada and North Western United States. (Hardy, 1963).

There are few reported cases where failure

*Indicates references listed at the end of this thesis.

occurred due to changes in the ambient solution of the soil. Recent publications which consider this aspect as definitely contributing to failure are Hardy (1963) and Bjerrum et al (1964). Hardy dealt with the case of slope failure where surface water entered the soil. The difference in concentration between the surface water and the solution already in the soil gave rise to an osmotic effect with resulting decrease in strength of the soil. Bjerrum et al reported the case of a tunnel failure near Oslo. Failure was caused by leakage through the joints of the concrete lining of the tunnel. The leaking water dissolved the carbonates of the adjacent material thus making space for the ingress of water. The material was montmorillonitic in nature and swelling occurred resulting in decreased shear strength. Analysis based on the excess hydrostatic pressure concept could not be used to account for the failure.

2.3 Explanations for the Swelling Phenomenon

If a dry sample of soil is exposed to an environment of increasing moisture conditions, (an atmosphere of controlled relative humidity) the soil will adsorb water molecules. The most recent research into this type of adsorbed water was reported by Barshad,

(1955). He used the mineral montmorillonite, and produced several adsorption isotherms. Curves for saturated sodium and calcium samples are given in FIG.1.

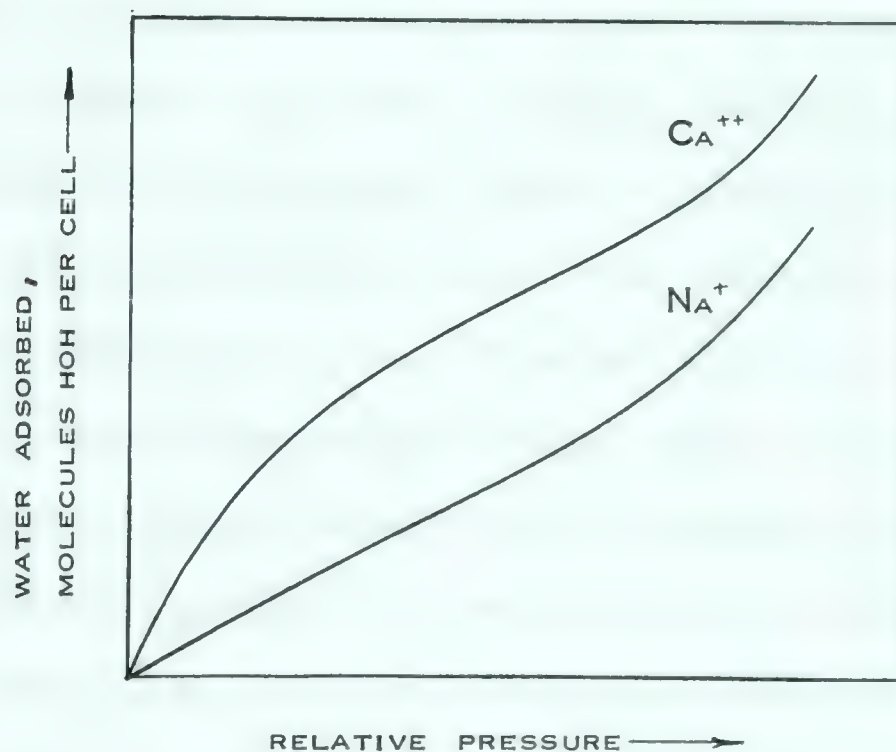


FIGURE 1: Adsorption Isotherm at 30°C of a Mississippi Montmorillonite Saturated with Sodium and Calcium Cations.

(After Barshad, 1955)

Barshad described the course of hydration and interlayer swelling of dehydrated and contracted montmorillonite as occurring in five distinct steps. The first step consisted of hydration of the external surfaces of the particles. On completion of the multimolecular layer of water on the external surface, the second step

consisted of a migration of water molecules to interlayer positions in the montmorillonite, equivalent to one layer of water molecules. The third step consisted of a reformation of the multimolecular water layer on the external surfaces of the particle. The fourth step consisted of another unilayer of water molecules migrating into the interlayer spaces. Expansion was accompanied by a redistribution of the already existing interior and external water molecules with the formation of a discontinuous dimolecular water layer in which water molecules were grouped around the interlayer cations. The fifth step consisted of a re-formation of the multimolecular water layer on the external surfaces of the particle. Step 5 was completed as a relative humidity of 100% was approached.

Barshad's work was based on observations using the X-ray diffraction technique. His findings were in agreement with previous researches done by Mattson(1932), Katz (1933), Marshall (1949), and McEwan (1955).

The amount of water adsorbed from the controlled atmosphere depends not only on the type of mineral comprising the clay particles but also on the type and amount of adsorbed cations. In samples saturated with cations of equal radius but varying charge the larger the charge

the lower the relative humidity at which expansion occurs. In samples saturated with the same cation but varying in amount the larger the number of cations the lower the relative humidity at which expansion occurs, (Barshad, 1955).

With sufficient time, a dry sample of soil exposed to a relative humidity of close to 100% will normally adsorb enough moisture to satisfy particle hydration and also cation hydration; if the sample is then brought in contact with liquid water more water will be taken in by the sample and further swelling will occur. This is the usual case encountered in soil mechanics practice. It is also the case for which the mechanism of water adsorption has been explained by more than one hypothesis. Some of these hypotheses will be briefly reviewed.

Sebastyan (1959) discussed the "Soil-Water Potential Theory" as related to swelling. This theory states that each solid particle has a certain water holding potential. A soil mass coming in contact with water will satisfy the water demand particle by particle. As the solid particles take in water, their unsatisfied water adsorption potential may be larger than the other external or internal force systems acting on the mass; the particles may be pushed apart by the adsorbed water

films and swelling occurs.

Another generally accepted hypothesis is the capillary effect. The most recent work utilizing this concept to explain the swelling phenomenon was presented by Clisby (1963). The argument presented in this case was that if a sample of moist soil were allowed to desiccate, capillary forces would develop. The capillary tensile forces would act normal to the walls of the capillary tubes in the soil. With evaporation of moisture, the capillary pressure would pull the particles together with resulting decrease in size of the soil mass. The process would continue until no further decrease in size occurred. The capillary menisci would then enter the wedges between the soil particles with further drying. If water should then be allowed to come in contact with the soil, water would be taken into the soil by capillary action and would destroy the high tensile stresses pulling the particles together. The release of these high pressures would then allow the soil particles to rebound thus causing swelling of the soil mass.

The "Osmotic Theory" is another method cited to explain the swelling phenomenon in soils. Osmosis is a dilution of a concentrated solution by a solution of lesser concentration, the two solutions being

separated by a semi-permeable membrane which permits flow of solvent but blocks the flow of solute, Maron & Prutton (1958). Osmosis is a mechanical process. It depends on the pore size of passages in the membrane along with the relative sizes of solvent and solute molecules. The process of osmosis can be explained in terms of the molecular kinetic theory. Consider a system consisting of a concentrated solution on one side separated by a semi-permeable membrane from a less concentrated solution on the other side as shown in FIG. 2.

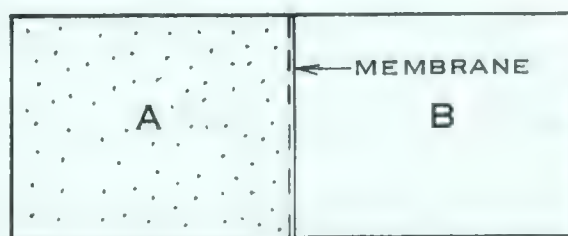


FIGURE 2 : Schematic Diagram to Illustrate the Development of the Osmotic Phenomenon.

The number of collisions of the solvent particles on the highly concentrated solution, (side A), with the membrane, will be less than that on the low concentration (side B). Therefore, the possibility exists for the number of solvent particles that will migrate from B to A to be more than those that will migrate from A to B. Hence, there will be a net movement of solvent particles from B to A, (Hiller and Herber, 1960).

The osmotic process depends on the number of these collisions for any solvent-solution system and will depend on the number of particles in the system. The pressure required on the more concentrated side to prevent a net migration of solvent particles through the membrane is the osmotic pressure. It is the number of these solvent particles and not their chemical properties that is used to develop the mechanism of osmosis. For this reason osmotic pressure is classified as a colligative property.

The application of the osmotic phenomenon to soils was discussed by Mattson (1933). He stated that the colloidal particle dissociates some of the adsorbed cations; the degree of dissociation depends upon the nature, valence and hydration of the ions. The dissociated ions form an atmosphere around the particle which in effect constitutes a highly multivalent colloidal ion of opposite charge to that of the diffusible ions. The concentration of the dissociated ions is greatest near the particle surface resulting in a steep concentration gradient perpendicular to the surface. The thickness of the ion atmosphere is limited by the electrostatic attraction. Within the ion atmosphere the osmotic pressure is greater than in the outside liquid and grows progressively greater in the direction of the particle

surface. The collidal particle in the ion atmosphere and the osmotically imbibed water constitute the colloidal clay micelle. The colloidal micelle can be represented as in FIG. 3.

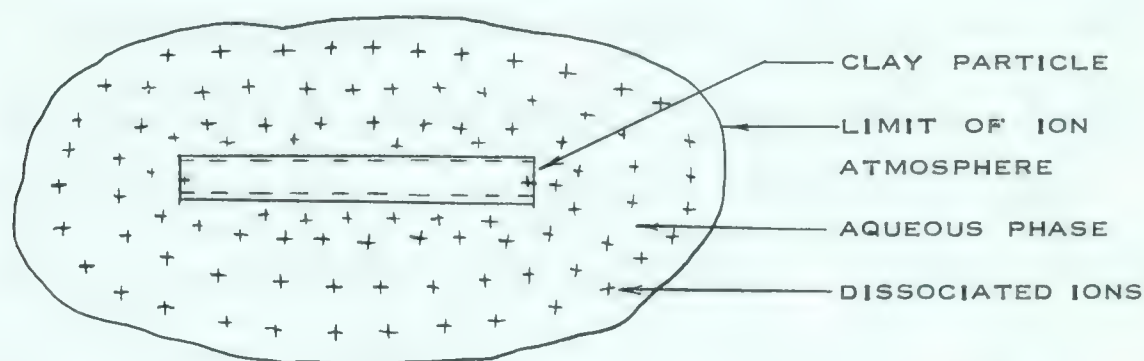


FIGURE 3 : Schematic Representation of a Clay Micelle.

If the moisture content of the sample is low then the aqueous phase of the micelle will be small and the ion atmosphere will be dense. Consider two adjacent colloidal micelles at low moisture content. The particles will be close together. If then the moisture content is allowed to increase, water will be taken in, limited by the capacity of the electrostatic forces, and the micelles will enlarge. This enlargement of the micelles results in expansion of the sample, in other words, the particles will be pushed apart. If, however, the sample is not allowed to expand, the tendency of the micelles to imbibe water will still exist; the pressure required to suppress this tendency to imbibe water and expand is essentially an osmotic

pressure and manifests itself as a swelling pressure.

The osmotic theory has until recently found limited application in engineering practice. The most recent interpretation of this concept was by Hardy (1964). Hardy considered the current concepts of the causes and effects of swelling in a clay soil. These concepts postulate the existence of attractive and repulsive forces between soil particles. The attractive forces are forces of molecular attraction or the Van der Waals forces. Besides the molecular forces which act over only short distances there are the electrical forces of attraction which act over a much greater distance. The repulsive forces due to hydration are essentially the same as found by Barshad for a dehydrated soil. Any further water adsorbed is then explained by Hardy as due to osmotic forces produced in the diffuse electrical double layer in the aqueous phase of the soil.

Hardy (1964), also points out that:

"The physico-chemical factors governing the development of osmotic swelling pressures in clay-water systems include the basic internal crystalline structure of the clay minerals, the geometric shape and size of the individual clay particles, the existence of a diffuse double layer surrounding each soil particle arising from the aqueous

environment, the nature and concentration of the exchangeable ions associated with the double layer, the nature and concentration of salts in the aqueous phase of the system, and the structure or fabric of the soil resulting from its geological history".

Attempts have been made in the past to relate the measured swelling pressure of a soil to the calculated osmotic pressure based on system characteristics. The equation used for calculating osmotic pressure was that developed by Van't Hoff:

$$P_s = RT \left[C_m + \frac{C_o^2}{C_m} - 2C_o \right] \dots\dots\dots 1$$

where, R = Universal gas constant

T = Absolute temperature

C_m = Concentration within the micelle

C_o = Concentration of the immersing liquid.

When the immersing liquid is water, C_o → 0, and the equation reduces to :

$$P_s = RT C_m. \dots\dots\dots 2$$

Bolt (1956) performed experiments using suspensions of sodium and calcium montmorillonite in dilute salt solutions. His observed results showed close agreement with calculations based on theoretical considerations. Similar agreements were obtained by

Warkentin et al, (1957), and Yong et al, (1962). These experiments were performed under ideal conditions such as, soil in suspension, minus 0.0002mm. particle size, low concentrations of pore water, and pure minerals.

The application of the Van't Hoff's equation to natural soils is not expected to show any quantitative agreement in measured and calculated pressures because the equation even as used in the ideal case is only an approximation. An attempt by Ruiz (1963) to directly relate these pressures for more natural soil conditions will be discussed in Chapter V.

Terzaghi (1956) in his discussion of Bolt's (1956) osmotic concept for explaining the swelling pressure developed in a soil, outlined the clay cluster concept. According to the clay cluster concept, a natural soil is composed of inert soil particles and of clusters of clay mineral particles forming minute osmotic cells. Terzaghi stated that the volume of the inert particles remains practically constant while the clay clusters are highly compressible and at a given pressure their size depends on the chemical composition of the soil solution. Therefore, if one should consider only swelling of the soil then Terzaghi's argument could substantiate the fact that osmotic pressure could contribute to volume change in a natural soil. The volume change would be

contributed by the clusters of clay particles since the inert particles would not undergo much volume change. Terzaghi's discussion was directed against Bolt's assumption that the mechanical behaviour of a soil was not "based on facts". Terzaghi pointed out that the mechanical properties of a soil influences the strength of a soil. He said that the shearing strength of each of the inert particles was equal to that of the mineral of which it consisted, whereas that of each clay cluster, parallel to its base, was equal to zero.

2.4 Previous related research done at University of Alberta.

P.A. Thomson (1960) attempted an investigation into the relative volume changes and the time-rate of volume change as a function of the adsorbed cations in samples of clay shales from the Peace River at Dunvegan. This investigation demonstrated the need for working with homionic samples. As a result, A.B. Hamilton, (1961), succeeded in preparing homionic samples of Lake Edmonton Clay. Consolidation tests were performed on these samples as well as on samples in which the exchangeable cations were controlled to comprise certain percentages of calcium and sodium cations.

G.J. Locker (1963), S. Thomson (1963) and A.E. Dahlman (1965), investigated strength characteristics

of these homionic samples. S. Thomson discussed some of the mechanisms of swelling. He considered the effect of the osmotic phenomenon on swelling and postulated that besides osmosis inherent with the clay micelle there might be a possibility of osmosis occurring between the soil mass itself and the immersing liquid. S. Thomson also demonstrated the fact that decreasing the concentration of salt in a soil resulted in a lower strength value. This he attributed to the fact that with the lower concentration in the soil solution more water could be osmotically imbibed. If, as a result of this, expansion was allowed with an increase in the moisture content of the soil, the compressive strength of the soil would decrease.

2.5 Mechanisms of Swelling Based on the Literature Review

The literature review reveals the following as possible mechanisms of swelling:

1. Particle hydration.
2. Cation hydration.
3. Water molecule adsorption by soil particles.
4. Capillary action.
5. Osmosis within the clay micelle.
6. Osmosis between the soil mass and the immersing liquid.

The elastic properties of the soil particles, in so far as swelling is concerned, can be considered a mechanism of swelling. However, rebound associated with elastic restitution of soil particles normally occurs almost immediately after load is removed and for this thesis, is not considered a problem in swelling soils.

Mechanisms of swelling should not be confused with factors that affect the degree of swelling in any one soil. A mechanism of swelling can be defined as a chemical or physical phenomenon which occurs in a soil resulting in expansion of the soil mass. Any condition that affects the degree of this chemical or physical reaction can be classified as a factor that affects swelling in a soil.

2.6 Factors that Affect the Degree of Swelling in a Soil

Some of the factors that affect the degree of swelling in a soil are:

1. The effect of the attractive Van der Waals forces.
2. Nature and amount of the adsorbed cations.
3. Nature and concentration of the pore water solution.
4. Structure of the soil.
5. Stress history.
6. Temperature conditions.

These factors complicate any investigation into the swelling behaviour of a soil. For this thesis, control of these factors had to be implemented, where possible, in order to study the mechanisms of swelling.

2.7 Summary

This chapter considered typical problems of swelling and the explanations for the swelling in soils as obtained from a literature review. The problem of swelling normally occurs when an expansive soil undergoes variation in moisture content. The manner in which the soil takes up moisture and then expands has been explained by particle hydration when the soil is initially in a dehydrated condition. For the condition when the soil is at moisture contents above the shrinkage limit (as is the case for most natural soils) the mechanism of swelling has been attributed to molecular attraction, capillary effect and osmotic imbibition. These postulates have been outlined.

CHAPTER III

SAMPLE PREPARATION AND TESTING METHOD

3.1 Introduction

The soil used for this investigation was a highly plastic glacial lake clay. It was the same soil as used by Hamilton (1961), Locker (1963), S. Thomson (1963) and Dahlman (1965). The soil was obtained from a fresh cut slope in South Edmonton at a depth of approximately 15 to 20 feet below original ground surface. After air drying, the soil was crushed to pass the No.40 seive. The soil was then treated to yield sodium and calcium homionic modifications. The other material used was pulverized Ottawa sand.

3.2 Outline of Testing Program

The object of the testing program was to investigate the mechanisms of swelling outlined in Chapter II. Some of these mechanisms as postulated, are inseparable from the soil science point of view since it is considered that the force contributing to water adsorption is the same regardless of the moisture content of the soil. However, for engineering purposes it is convenient to treat the mechanisms separately and to

consider their effects on soil swelling behaviour, individually, but in so doing it must be realized that no distinct boundaries exist between one mechanism and another. It was therefore intended to investigate the mechanisms not only to determine their contribution to soil swelling but also to indicate the conditions under which they predominate with regard to soil moisture and stress considerations.

The testing program was as follows:

(a) Particle Hydration

This mechanism is considered to occur as a result of an attraction of water molecules by the soil particles when the moisture condition of the soil is such that the layers of adsorbed water around the particles are not fully developed. To investigate this mechanism tests were performed using soil samples at initial moisture contents of approximately 1 percent.

(b) Cation Hydration

It was intended to investigate the effect of cations on soil swelling by testing the sodium and calcium soils at low moisture contents as for (a) above.

(c) Attraction of the Soil Particles for Water Molecules (Soil water Potential Theory)

The explanation of soil swelling by this theory is basically similar to the concepts of the

soil scientists. However, the theory as presented, (Sebastyan, 1959), does not consider variation in swelling behaviour when a salt solution is in contact with the soil instead of water. It is intended to point out this limitation by testing the clays in water and in salt solutions.

(d) Capillary Effect

The capillary effect is postulated as a mechanism of swelling based on the fact that when the capillary menisci pulling particles together are destroyed the sample rebounds. It was intended to investigate this aspect of swelling by testing the clay material in varsol. Varsol was used because it is non-polar but has a surface tension approximately 25% that of water. With these properties it was considered that capillarity can be developed while other complications, for example osmotic effect, are reduced. It was also intended to consider the results on the pulverized sand sample in which water is held mainly by capillarity.

(e) Osmotic Action Between the Adsorbed Water Layer and the Pore Water

This osmotic effect can be considered to occur between the solution in the adsorbed water layer (double layer) and the pore water solution, as a result of a difference in concentration between the two

solutions. It was intended to investigate this osmotic effect by testing the sodium and calcium clays in water and in salt solutions of varying concentrations, with the samples at various initial moisture contents.

(f) Osmosis Between a Soil Mass Containing Saline Solution and an External Solution of Lesser Concentration

This postulate is based on the fact that as a result of the saline solution in the pores of the soil mass, an osmotic action can be developed if the soil comes in contact with a solution of lesser concentration. This mechanism was investigated using the pulverized sand with salt added.

3.3 Homionic Preparation

The method of preparing the homionic samples of sodium and calcium clays was essentially the same as outlined by Hamilton (1961). Slight deviations from the original procedure were undertaken as a matter of convenience. It was estimated that about 1 kilogram of each modification was required hence 1.5 kilograms of the natural remoulded soil were used. This amount of soil was placed in a 4-gallon polyethelene pail and treated with four batches, each of 3,750 cc, of 0.75N hydrochloric acid. Each application of the hydrochloric acid solution was stirred in by hand using a wooden

stirrer for approximately 10 minutes. The slurry was left to settle out. After about 24 hours the supernatant liquid was syphoned off. This hydrochloric acid treatment was found satisfactory for the removal of the carbonates and sulphates in the soil. The removal of these salts was considered necessary to prepare homionic soil samples. The carbonates are slowly soluble in water and this condition gives rise to the undesirable possibility of a release of extraneous cations in the soil. The presence of these salts lowers the cation exchange capacity of the soil. The salt crystals occupy space in the soil thus decreasing the amount of clay particles in any given volume of soil.

The wet soil was next washed with four batches, each of 3,750 cc, of distilled water. The washing procedure was the same as for the hydrochloric acid. Washing with water was applied in order to remove the excess acid in the soil.

The soil was then saturated with the acetate solution of the desired modification. Five batches of 3,750 cc. each of 1.0N acetate solution were used. Each batch was applied as for the hydrochloric acid treatment. This acetate solution treatment was necessary in order to force the clay particles to accept the

particular type of cation desired in exchange for the various types of adsorbed cations in the natural soil. Acetate salts were used because of the ease with which the acetate radical can be later removed from the soil.

The next operation was the removal of the excess salts from the soil samples. This can be done by washing with either pure water or ethyl alcohol. The alcohol was preferred for three reasons. Firstly, the acetate is soluble in alcohol and can therefore be easily removed. Secondly, water will cause some replacement of the adsorbed cations by hydrogen ions present in the water thus lowering the number of positions filled with the desired cations. When alcohol is used, this situation does not arise. Thirdly, alcohol being less polar than water, facilitates the washing and centrifuging process.

Washing was accomplished by placing approximately 40 grams of the wet soil in a 200 cc. centrifuge bottle. About 75 cc. of alcohol was then stirred in for 2 minutes using a rubber tipped stirrer with the milk shake machine. The soil was centrifuged. The clear solution was poured off and about 25 cc. of it were oven dried to check on the presence of salt crystals. This washing procedure was repeated about seven times when evaporation of the supernatant liquid showed that the presence of salt crystals approached zero. The centrifuging time at

approximately 2000 r.p.m. ranged from 10 minutes for the first washing to 45 minutes for the last washing.

After washing out the excess salts, the soil was extracted from the centrifuge bottle and oven dried at 60°C. The soil was then pulverized to pass a No.40 seive.

To obtain samples of silica sand of comparable grain size as the clay material, Ottawa sand was pulverized. This was done by the Department of Mining and Metallurgy at the University of Alberta. About 500 grams of sand were pulverized by both dry and wet processes. Crushing of the particles was effected by impact with small porcelain balls. The wet processed material was used as it showed the finer grain size as determined by the hydrometer analysis. The wet processed material had 35% finer than .002 mm. as compared with 45% for the natural Lake Edmonton clay. However, the nature of the results required from the tests on the sand indicated that the grain size obtained was satisfactory.

3.4 Sample Characteristics

Classification tests were carried out on the clay and its sodium and calcium modifications. Tests on these samples for total exchange capacity and cation exchange capacity, were in accordance with the procedures

of the Alberta Soil Survey. The mineralogical composition of the clay was determined by the Soil Science Department, University of Alberta, using the X-ray diffraction technique. The results of the above tests on the clay are tabulated in TABLE I. The specific gravity and hydrometer analysis tests were done on the pulverized sand sample and the results are also reported in TABLE I. An attempt to determine the liquid and plastic limits of the pulverized sand was abandoned owing to the high dilatency of the material. The sample was mixed by hand at a moisture content of 20%. It displayed a preference for bulking but the lumps were easily crushed with the fingers. The soil then was a moist powder. The moisture content was increased to 24% but the soil exhibited the same behaviour as at 20%, that is, it still was a moist powder when mixed with the hands. The container was now tapped with the hands and the soil changed its properties from a moist powder to a viscous liquid. The soil was again mixed with the hands to the moist powder condition and some of this material was put in the liquid limit machine. With the first two blows the soil behaved as a dry material but by the fifth blow the remaining soil had liquefied. It was, therefore, impossible to mix and place the soil for a liquid limit determination. It was similarly difficult to obtain a

TABLE I: SAMPLE CHARACTERISTICS

SAMPLE	UNITS: M.E. PER 100 GRAMS OF DRY SOIL													
	* w _L	w _P	I _P	w _S	G _S	%SIZES LESS THAN 2 μ	EXCHANGEABLE CATIONS				ΣE.C.	* T.E.C.	SALTS	
							CATIONS							
							Na ⁺	Ca ⁺⁺	Mg ⁺⁺	K ⁺				
I	2	3	4	5	6	7	8	9	10	11	12	13	14	
NAT. REM. L.E.C.	74.3	30.8	43.5	14.6	2.79	46	1.0	93.0	2.5	0.5	97.0	27.5	69.5	
NAT. REM. [PORE WATER]							3.2	84.5	5.1	1.5	94.3			
SODIUM	105.8	27.4	78.4	15.0	2.83	66	33.4	0.5	0	0.3	34.2	31.0	3.2	
CALCIUM	74.0	32.3	41.7	16.3	2.73	48	0	33.0	3.5	0.3	36.8	31.9	4.9	
PULVERIZED SAND	[22-23]	[22-23]	N. P.		2.66	35								

MINERALOGICAL COMPOSITION OF THE CLAY SIZE FRACTION OF THE NATURAL REMOULDED CLAY ***

MONTMORILLONITE	50 TO 55%
ILLITE	27 TO 32%
CHLORITE	10 TO 20%
KAOLINITE	10 TO 20%

[NOT DISTINGUISHABLE]

* SEE GLOSSARY OF TERMS AND SYMBOLS

** BY DISTILLATION

*** DR. S. PAWLUK, DEPARTMENT OF SOIL SCIENCE, PERSONAL COMMUNICATION.

plastic limit determination because depending on the rate of rolling and pressure applied the soil could be made into a powder or thin thread with the soil at the same moisture condition. Based on the moisture contents used, the liquid limit can be estimated at 22% to 23% with the plastic limit being about the same but slightly less than the liquid limit.

3.5 Sample Preparation

For the constant volume one-dimensional tests the clay samples were to be tested at varying initial moisture contents. In order to attain approximately the same structural arrangement of the soil particles, the soil was mixed at a moisture content approximating the liquid limit. The soil was then covered and placed in the moist room to soak for about 1 day. The soil was then carefully placed in a 3 inch diameter by $\frac{1}{2}$ inch thick lucite ring, in thin lifts, to avoid entrapping air bubbles. A moisture content determination of the soil, as placed in the ring, was made. This allowed checking the moisture content of the sample during the desiccation process. The sample in the ring was kept in a large covered chamber at first. When the sample had shrunk slightly in the ring after about 1 day, it was placed in a covered dish and kept in the chamber.

This was done in order to lower the rate of desiccation and to prevent cracking of the sample. Periodic checks were made to trace the moisture content drops of the sample. It was thus possible to obtain samples at any desired moisture content. It was found that once a required moisture content was reached the sample could be wrapped in a plastic bag without entailing significant moisture loss for days. This enabled tests to be started as convenient.

For the test on the clay using varsol the sample was mixed in a proportion of 50 grams of soil to 50 cc. varsol. This resulted in an excess of varsol but this condition was desired in order to place the soil in the testing ring as a liquid thereby decreasing the possibility of entrapping air. As the slurry was poured in the ring, the free varsol drained out. The soil was then levelled in the ring and was ready for testing.

The pulverized sand exhibited a bulking behaviour and high dilatency. From a consideration of the results required from this test it was decided that the most feasible way to test the material was to place the dry material loosely in the testing ring, compact it in the consolidometer and then allow access to water from below the sample. It was expected that as the

water migrated upwards into the sample most of the air in the sample will be pushed up and out.

3.6 Testing Method

The tests were mainly to determine swelling and swelling pressures. For this reason the one-dimensional consolidation test was used; in particular, the constant volume one-dimensional consolidation test. The constant volume test was chosen because in this way the swelling pressure of the soil is measured without allowing the soil itself to undergo any major change in characteristic. In addition, the constant volume test requires a shorter period of time for completion. However, both the free swell and constant volume tests have applications to particular field problems, (Hardy and Dyregrove, 1962).

The consolidation machines were of the platform type with a mechanical advantage of 100 and a capacity of 25 tons/ft² using 2.5 inch-diameter samples. In addition one large consolidation machine was used. It has a mechanical advantage of 200 and a capacity of over 150 tons/ft² using 2.5 inch-diameter samples. The accuracy of the machines was checked against a calibrated proving ring.

The consolidation cells, specially made for the tests, were of plastic material based on the design

of the existing metallic cells. The porous stones were corundum instead of bronze. These materials were used to eliminate all metal contacts with the soil during the test in order to avoid any possibility of ion exchange with metals. The consolidation rings were of lucite with a polished inside surface. The rings were nominally 2.5 inch inside diameter and 1.25 inches deep. The rings were checked for circumferential expansion using a wet sample of soil and maximum load. Expansion was less than 0.01 mm. which for this program was considered insignificant. With regard to side friction, it was not considered necessary to apply a correction because the polished surface when coated with high vacuum silicone grease reduces the friction effects to a rather small value, (Brooker, 1964). The low friction between soil and ring was evidenced at the end of the test when the samples could be extracted from the ring with little or no effort.

Compressibility of the consolidation apparatus and filter paper was significant. A metal plate had to be placed under the base of the consolidation cell to avoid bending of the cell and breaking of the porous stones. The apparatus had to be calibrated for deformation of the cell, filter paper and plastic loading cap as well as warping of the platform. These deformations

had to be taken into account as the test progressed in order to study the deformation in the soil sample itself.

In the constant volume consolidation test the samples at the required moisture content were trimmed to fit the consolidation rings using a plastic cutter. The trimmings were used to check on the moisture content of the sample. For the drier samples, it was necessary to sand paper the sample to size on a sand paper machine. (The initial moulding size of the sample was so chosen that the required size sample could be obtained with only a small amount of trimming). The volume and weight of the sample were determined. The sample was then inserted in the consolidation ring with a minimum loss of time and was set up for loading. The immersing liquid was put into the consolidation cell and loads were applied to prevent expansion of the sample with due consideration given for deformation inherent with the apparatus itself for each load. At least two increments of loads were used beyond the point when the sample commenced consolidating. After consolidation under the final load the sample was allowed to rebound according to the procedure of the standard consolidation test.

For the clay test using varsol as the immersing liquid, and for the test on the pulverized Ottawa sand the method of placing the sample in the consolidation cell was previously described. The loading process for these tests were in accordance with the normal procedure for standard consolidation tests.

3.7 Summary

This chapter gave an outline of the testing program, a description of the samples, their preparation, the apparatus and the testing method. The soil samples were Lake Edmonton clay with its sodium and calcium modifications, and pulverized Ottawa sand. The characteristics of the samples were tabulated in TABLE I. The apparatus was the platform consolidometer using a non-metallic consolidation cell. The swelling pressures on the clays were determined by the constant volume one-dimensional consolidation test at varying initial moisture contents of the samples obtained by desiccation. Standard consolidation tests were run on the pulverized Ottawa sand and on a clay sample using varsol as the immersing liquid.

CHAPTER IV

TEST RESULTS

4.1 Introduction

In this chapter the results of the tests will be presented and explanations will be offered concerning the nature of the test results. These results will be used in Chapter V to discuss the mechanisms of swelling, which was the fundamental purpose of this investigation.

The samples were tested at varying initial moisture contents and in turn with varying immersion solutions. Because of these conditions different maximum loads were applied to the samples. This had an effect on the amount of swell obtained for any sample based on net volume change between initial and final conditions. Consideration was given to this problem and a discussion will be presented on the amount of swell for samples having approximately the same maximum stress.

For the purpose of this investigation, the swelling pressure is defined as the pressure at which the sample revealed a definite decrease in volume compared to initial conditions. In view of the difficulty of maintaining a constant volume of the sample during the test,

the sample was subjected to small increases and decreases in volume before consolidation started. The maximum pressure at which the consolidation curve cuts the no volume change line was defined as the swelling pressure. (See FIG. 4).

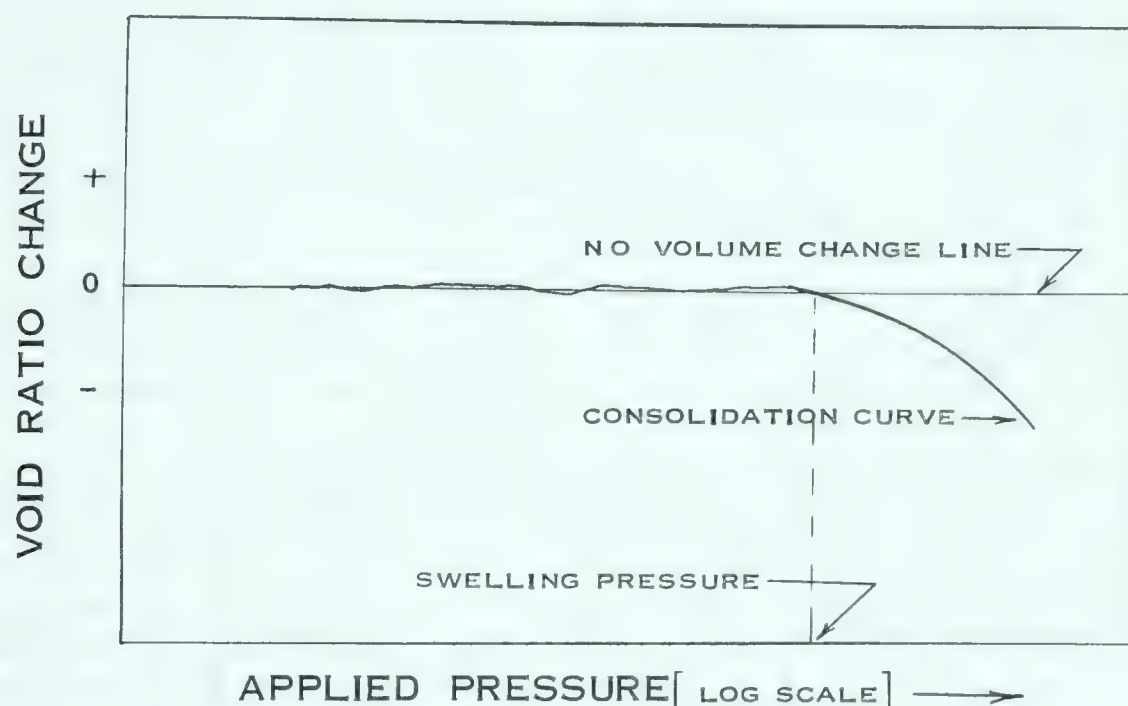


FIGURE 4: Definition of Swelling Pressure.

4.2 Test Results

Tests were carried out on natural remoulded Lake Edmonton clay, calcium and sodium modifications, and pulverized Ottawa sand. The results of the tests are summarized in TABLE II. Void ratio versus pressure plots are included in Appendix A.

The following notes are applicable to TABLE II:

1. Of the 31 tests reported, Nos. 13 and 25

TABLE II SUMMARY OF TEST RESULTS

TEST No.	SAMPLE	SOLUTION	H _r *	AREA CM ²	W _s GMS	W _i %	W _f %	S _{RI} %	e _i	e _f	P _{MAX} KG/CM ²	P _{MIN} KG/CM ²	P _S KG/CM ²	%ΔV	C _C	C _S	ACTIV-ITY
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18
1	L.E.C.	HOH	0.354	32.6	31.38	54.2	39.1	94.2	1.596	1.0910	9.23	0.0039	0.12	-17.91	0.6	0.18	0.95
2	L.E.C.	HOH	0.382	33.1	46.64	28.6	39.5	84.5	0.93	1.1021	18.1	0.0085	1.8	7.38		0.10	
3	L.E.C.	HOH	0.363	33.1	51.46	14.1	38.0	60.0	0.6546	1.0602	18.1	0.0087	20.5	26.64		0.14	
4	L.E.C.	HOH	0.330	32.6	51.50	1.85	39.9	9.6	0.540	1.0853	55.2	0.0125	17.5	33.78		0.15	
5	L.E.C.	VAR SOL	0.553	33.1	48.25				1.6883	1.1195	18.1	0.0087	-	-21.2	0.30	0	-
6	CALCIUM	HOH	0.250	33.1	23.30	52.9	42.8	98.9	1.4633	1.1785	18.1	0.0085	0.65	11.56	0.54	0.14	0.87
7	CALCIUM	HOH	0.255	33.1	29.82	30.7	42.1	86.8	0.9778	1.2048	30.02	0.0085	5.3	11.57		0.10	
8	CALCIUM	HOH	0.283	33.1	38.21	21.3	41.6	83.1	0.7000	1.2082	30.02	0.0085	17.0	29.89		0.19	
9	CALCIUM	HOH	0.270	32.6	36.93	16.7	38.9	72.72	0.6008	1.1300	73.6	0.0125	33.0	28.89		0.14	
10	CALCIUM	HOH	0.262	32.6	38.73	1.0	37.5	5.15	0.5293	1.0786	73.6	0.0125	30.0	35.92		0.17	
11	CALCIUM	0.5 M CA AC ₂	0.338	32.6	32.32	50.5	40.7	100.	1.3641	0.9990	18.4	0.0078	0.04	-15.44	0.49	0.08	
12	CALCIUM	0.5 M CA AC ₂	0.231	32.6	27.74	29.0	39.6	89.7	0.8827	1.0824	30.7	0.0078	2.0	10.61		0.09	
13	CALCIUM	0.5 M CA AC ₂	0.265	32.6	38.15	17.0		72.4	0.6313		36.8		28.0				
14	CALCIUM	1.75 M CA AC ₂	0.358	33.1	33.55	52.4	35.54	99.65	1.4492	0.9847	18.1	0.0087	0.015	-18.97	0.59	0.135	
15	CALCIUM	1.75 M CA AC ₂	0.320	32.6	40.53	28.1	35.49	97.78	0.7848	1.0610	30.7	0.0088	1.4	17.03		0.10	
16	CALCIUM	1.75 M CA AC ₂	0.256	32.6	37.25	16.3	34.76	80.48	0.5528	1.2901	73.6	0.0125	30.0	47.46			
17	CALCIUM	1.75 M CA AC ₂	0.265	32.6	38.85	1.0	30.57	5.17	0.5418	1.0369	73.6	0.0125	33.0	32.11		0.10	
18	SODIUM	HOH	0.342	32.6	30.23	53.6		91.63	1.6633				1.3				1.19
19	SODIUM	HOH	0.274	33.1	32.75	31.9		94.65	0.9201				9.2				
20	SODIUM	HOH	0.270	32.6	36.84	21.8		85.94	0.7175				14.0				
21	SODIUM	HOH	0.240	32.6	35.97	16.3		76.82	0.5864				31.0				
22	SODIUM	0.5 M NA AC	0.335	32.6	29.65	55.0	44.20	94.47	1.6523	0.9576	18.4	0.0089	0.074	-26.24	0.65	0.25	
23	SODIUM	0.5 M NA AC	0.293	33.1	35.09	32.1	43.89	92.07	0.9866	0.9459	30.02	0.0085	2.2	-2.05		0.11	
24	SODIUM	0.5 M NA AC	0.290	32.6	38.99	22.7	41.91	86.47	0.7430	1.1615	30.7	0.0078	11.4	24.00		0.13	
25	SODIUM	0.5 M NA AC	0.240	32.6	34.20	16.5		76.82	0.5864		36.8		28.0				
26	SODIUM	0.5 M NA AC	0.249	32.6	38.42	1.12	38.17	6.10	0.5196	1.2397	73.6	0.0125	31.0	47.39			
27	SODIUM	4 M NA AC	0.340	33.1	32.69	50.0	34.72	95.98	1.4742	1.1474	18.1	0.0087	0.03	-13.21	0.66	0.08	
28	SODIUM	4 M NA AC	0.230	32.6	27.88	31.0	28.35	94.02	0.9331	0.9339	18.4	0.0078	1.4	0.43		0.02	
29	SODIUM	4 M NA AC	0.250	32.6	39.28	1.1	23.17	6.83	0.4554	0.9858	73.6	0.0125	33.0	35.56		0.11	
30	SAND	HOH	0.503	32.6	48.88	38.82	32.7	95.47	1.0814	0.9153	18.4	0.0078	-	-7.35	0.05	0	N.P.
31	SAND	HOH	0.520	32.6	48.74	57.54	38.25	100.	1.3897	1.0110	18.4	0.0078	-	-14.98	0.15	0	

*SEE GLOSSARY OF TERMS AND SYMBOLS

were tested for swelling pressures only.

The rebound curves were not determined.

2. In Test No. 16, after the sample was consolidated under maximum load (73.6 tons/ft^2), the sample was unloaded directly to minimum load (0.0125 tons/ft^2) to note the rebound characteristics under drastic unloading conditions.
3. Tests 18 to 21 representing the sodium samples tested in water could be tested only for swelling pressure. Attempts to consolidate the samples beyond the swelling pressure was not considered worthwhile as with elapse of time the clay particles started to migrate into the immersion water through the clearance between the porous stones and the consolidation ring.

4.3 Identification of Swelling Clays

Holtz and Gibbs (1956) in searching for simple identification tests for expansive soils found that the colloid content of a soil and the Atterberg limits could provide satisfactory indicators of the expansive characteristics of clays when considered together. A critical examination of the data they presented to substantiate

their findings would indicate that their conclusions could be misleading. Their charts show that for any one value of the index used (colloid content, plasticity index, or shrinkage limit), soil types exist which occupy a range from a classification of low to very high expansive characteristics. However, this method of identifying expansive soils is quite simple and finds wide practical use. More elaborate methods such as X-ray diffraction, differential thermal analysis, and flame photometric analysis, though they present better results on identification of soils, their practical application to routine testing is severely limited by economic and time considerations.

Seed et al (1962a) realizing the limitations of the identifying method proposed by Holtz and Gibbs, pursued studies on compacted materials and introduced the swelling potential as a means of identifying swelling characteristics of any soil. The swelling potential is defined as the percent swell of a laterally confined sample on soaking under 1 p.s.i. surcharge after being compacted to maximum density at optimum water content in the standard AASHO compaction test. Seed et al also reported that though two soils may have the same swelling potential they may exhibit quite different amounts of swell after compaction due to variation in

placement composition, manner of compaction, or to differences in climatic conditions. Therefore, though a soil may be identified as being of the expansive type based on the swelling potential, the amount of swell and swelling pressure can only be obtained from appropriate tests.

The concept of activity introduced by Skempton (1953) can also be used for identifying expansive soils. This method makes use of the plasticity index and percent clay sizes, hence, it will have similar limitations as the method proposed by Holtz and Gibbs.

TABLE III lists the classification of the soils used in this program according to the three identifying methods:

TABLE III : Classification of Soils

Sample	Holtz & Gibbs	Swelling Potential	Activity
Rem.L.Edm.Clay	Very High	High	Normal
Sodium	Very High	Very High	Normal
Calcium	Very High	High	Normal
Pulverized Sand	Low	Low	Insensitive

TABLE III shows that identifications based on activity and Holtz and Gibbs' method give the same classification for the remoulded Lake Edmonton clay, calcium and sodium

modifications. The method of Holtz and Gibbs indicates that these soils are very high swelling types, while they are classified as normal soils based on activity alone. The swelling potential classification chart developed by Seed et al relates the activity and percent clay sizes, with indicated ranges of swelling potential. This chart gave the best indication as to the actual behaviour of the samples as tested. It showed the sodium sample to have very high swelling characteristics as distinct from the remoulded Lake Edmonton clay and calcium samples, which were shown as having high swelling characteristics. All methods indicated that the sand sample was a low swelling type.

The results of the consolidation test can also be used to identify expansive soils in terms of the ratio, $a = \frac{C_s}{C_c}$, where C_c is the compression index and C_s the swelling index. The higher the value of "a" the greater will be the expansive characteristic of the soil. From the test results both the natural Lake Edmonton clay sample and the calcium sample, tested in water, gave a value of 0.3 indicating that these samples had similar swelling characteristics. A value for "a" could not be obtained for the sodium sample tested in water. However, the results obtained by Hamilton (1961) showed that the sodium clay had a value of 0.8 for "a" indicating that

the sodium sample had a greater expansive characteristic than either the calcium clay or the remoulded Lake Edmonton clay. When the calcium clay was tested in a 0.5M salt solution the value of "a" was 0.16 as compared to 0.3 when tested in water, indicating that with a salt solution in the pore water the soil will exhibit a lower swelling characteristic.

4.4 Percent Volume Change

The problem of predicting the amount of swell is much more complex than determining whether the soil is of the expanding type. The test results show that for the same soil composition and approximately similar structure, (particle arrangement), wide variations in amount of swell can be obtained depending upon the initial moisture content of the sample, the stress history and the nature of the surrounding solution.

Consider the Lake Edmonton clay and calcium samples tested in water, TABLE IV. The tabulated value show that volume change increases as the initial moisture content decreases. This is to be expected since with a lower initial moisture content the soil will have a greater capacity to imbibe water. An apparent discrepancy appears to occur with test No. 9 but the lower value of

TABLE IV : Initial Moisture Content vs
Volume Change

Test No	Sample	Pmax	ω_i %	% Vol.Change
1	L.E.C.	9.23	54.2	-17.91
2	L.E.C.	18.1	28.6	+ 7.38
3	L.E.C.	18.1	14.1	+26.69
4	L.E.C.	55.2	1.85	+33.78
6	Calcium	18.1	52.9	-11.56
7	Calcium	30.0	30.7	+11.57
8	Calcium	30.0	21.3	+29.89
9	Calcium	73.6	16.7	+28.89
10	Calcium	73.6	1	+35.92

the percent volume change for this test with initial moisture content of 16.7% as compared with test No.8, is due to the higher consolidation pressure applied to the sample of test No. 9. This introduces the influence of the stress history on the percent volume change.

The influence of the stress history on percent volume change can be studied by considering the samples for tests Nos. 1 and 6, which showed swelling pressures of 0.12 and 0.65 tons/ft² respectively. If the samples were allowed to swell as in a free swell test, a positive value of volume change would be obtained, but by subjecting

the samples to consolidation pressures higher than the swelling pressure, a net reduction in volume was obtained. This was due to a permanent set that occurred in the sample on first loading, as a result of which the sample could not regain its original volume.

Clisby (1963) has shown that by repeatedly loading and unloading the soil the permanent set decreases with each cycle of loading. With application of sufficient cycles of loading, Clisby claims that the soil can be made to behave elastically. Elastic behaviour in this sense being considered as the compressibility and expansibility of the soil mass including expulsion and imbibition of water, compression and expansion of air voids and soil solids. Hence the total rebound is sometimes said to be due to elastic rebound. However, on the basis of physico-chemical concepts soil swelling can be the result of more than one cause. Elastic rebound in the physico-chemical sense as used in this thesis is somewhat different from the concept of Clisby (1963) or Skempton (1960), in that expansion due to osmotic and particle hydration forces are not included as elastic rebound.

Clay particles hold water at varying electrical potentials depending on the distance from the particle surface, as shown in FIG. 5.

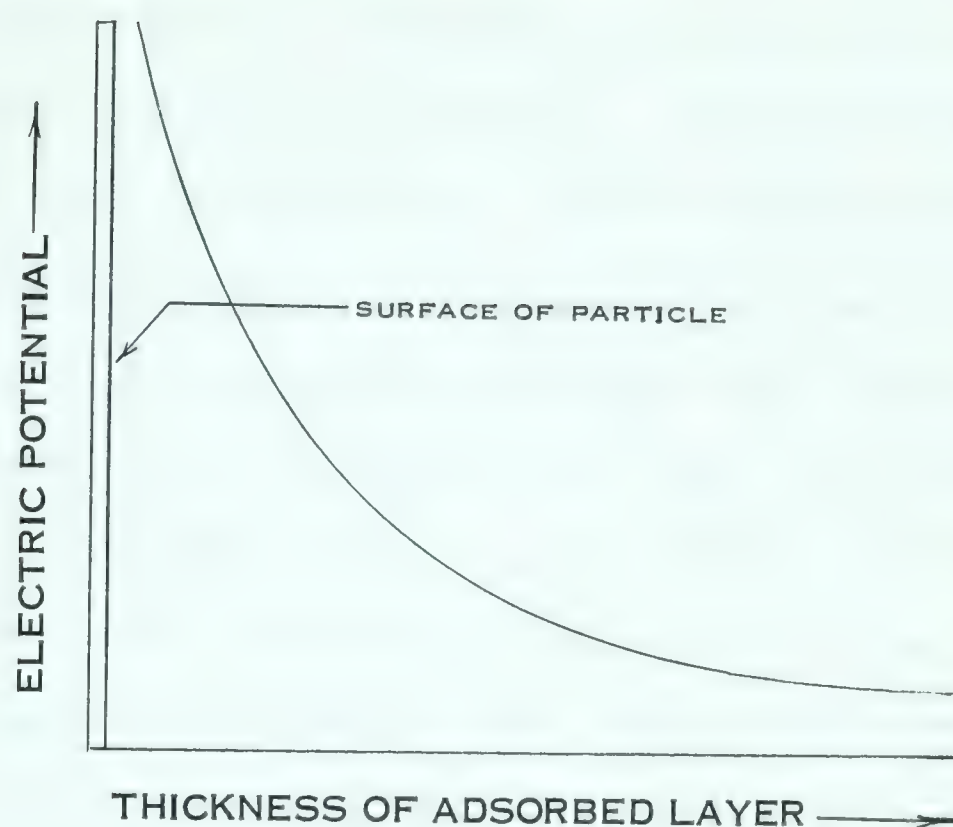


FIGURE 5: Potential States and Thickness of the Adsorbed Water Layer.

The potential is higher near the surface of the particle and decreases exponentially with distance from the particle surface. When the load on the sample is low each particle will have a thick adsorbed water layer and the potential at the outside edge of the adsorbed layer will be low. When the sample is loaded to higher pressures the soil particles will release some of their adsorbed water resulting in a thinner adsorbed water layer. The potential state of the clay micelle will correspondingly increase to be in equilibrium with the applied pressure. This phenomenon simulates the osmotic effect. When the

load on the sample is decreased, the clay particles with adsorbed water can exist at a lower potential state and water that was previously released can be imbibed.

When the soil is in contact with a salt solution, the clay particles with the adsorbed water layer undergo an adjustment to be in equilibrium with the outside solution. This means that a soil existing in an environment of low salt concentration if exposed to an environment of high salt concentration, will release some of the adsorbed water in order to attain a state of higher potential to be in equilibrium with the potential of the outside solution. Whether or not the soil mass decreases in volume as a result of the reduction in the adsorbed water phase, depends on the stresses applied to the soil. On the other hand, a soil existing in an environment of high salt concentration, if exposed to an environment of low salt concentration, will have the excess salts in the soil partly removed by dilution. This reduction in concentration in the free water phase allows the clay micelles to exist at a lower potential state, hence, the clay micelles will imbibe water and expand. This phenomenon also is essentially an osmotic effect.

The effect on percent volume change due to the concentration of the solution in contact with the soil was

demonstrated in the testing program. Comparable results are presented in TABLE V.

TABLE V : Effect of Solution Concentration on Percent Volume Change.

Test No.	Soil	Solution	$\omega_i\%$	Pmax	%Vol.Change
6	Calcium	HOH	52.9	18.1	-11.56
14	Calcium	1.75M Ca Ac ₂	52.4	18.1	-18.97
25	Sodium	0.5M Na Ac	1.12	73.6	+47.39
28	Sodium	4.0M Na Ac	1.1	73.6	+35.56

These results are chosen because in each set of samples, calcium and sodium, the samples had similar initial moisture contents and stress history. The percent volume change for each set is shown to be lower for the samples tested in the solution of higher concentration.

Another factor that influences the amount of swell a soil exhibits is the nature of the adsorbed cations. It is well known that with sodium as the adsorbed cations the soil will swell considerably more than if calcium cations were the adsorbed ions. This condition was demonstrated during testing. The sodium clay when tested in water had such a high capacity for imbibing water that the

soil particles began dispersing into the immersing solution. One of the sodium samples was left in the consolidometer for several days. On removal, the remaining soil was in a state of a slurry, the moisture content being well over 100%. Had the sample not disintegrated, the amount of swell for the sodium clay would have been considerably more than that for the calcium clay.

4.5 Moisture Pick Up

TABLE II shows that the Lake Edmonton clay samples tested in the range of initial moisture contents and varying maximum consolidation pressures, attained about the same moisture content at the end of testing. On the other hand, the calcium and sodium samples consistently showed a decrease in final moisture content, as the initial moisture content decreased and in turn as the concentration of the testing solution increased.

TABLE I shows the excess salts to be high (69.5 m.e. per 100 grams a.d.s.) in the case of the Lake Edmonton clay while in the case of the sodium and calcium samples the salt content was low (3.2 and 4.9 m.e. respectively). When the soil samples are desiccated or consolidated the particles are brought closer together. In some cases two or more particles may be

joined temporarily and later behave as a single particle. The capacity to imbibe water will therefore decrease. The joining of particles is likely to occur to a greater extent in the cases of sodium and calcium samples, than in the case of the Lake Edmonton clay. Upon drying of the Lake Edmonton clay sample, the excess salt may crystallize and occupy positions between soil particles. This salt goes into solution when water again comes in contact with the sample. The number of particles will remain essentially the same and the capacity to imbibe water will be unaltered.

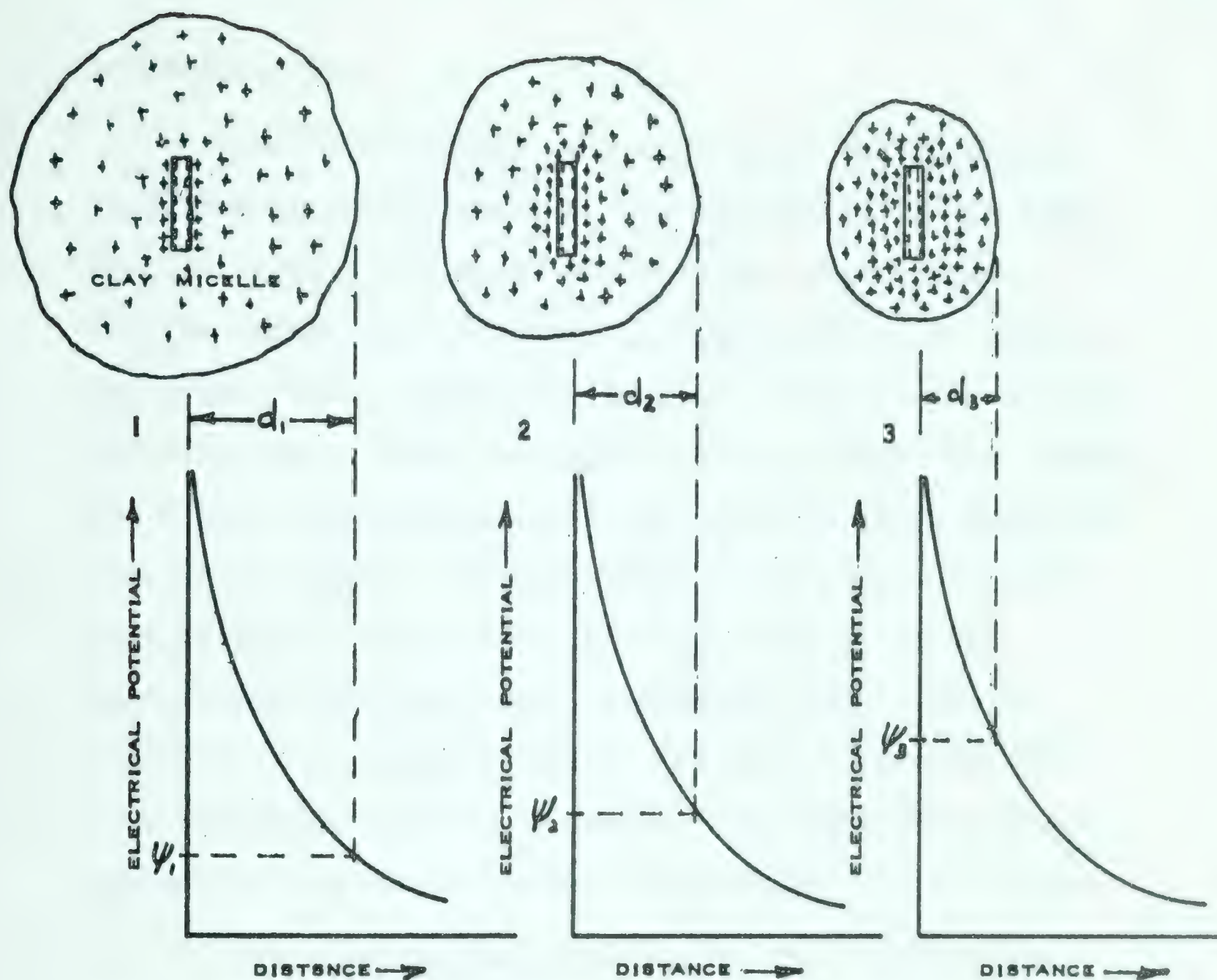
The lowering of the final moisture content with increasing salt concentration in the pore water was expected. With higher concentration in the pore water the clay micelles exist at a higher potential to maintain equilibrium. To achieve this condition the water phase must be small, hence lower moisture contents of the samples will prevail.

4.6 Swelling Pressures

If a soil has the capacity to swell, it will expand when in contact with water. If the expansion is to be suppressed then a pressure must be applied. It has also been shown that the swelling tendency can be partially suppressed by allowing the soil to come in

contact with a solution higher in concentration than the pore water. When the above mentioned conditions prevail concurrently, the clay micelles will attain a state of equilibrium with both the environmental solution and the applied pressure. This is achieved by successive adjustment of the adsorbed water as illustrated in FIG. 6.

In case (1) of FIG.6 the soil is in equilibrium with water at low potential, ψ_1 . The thickness of the adsorbed layer is d_1 . If now the soil comes in contact with a solution having a potential, ψ_2 , then the clay micelle reduces in size to thickness d_2 to attain the higher potential, ψ_2 . If this high concentrated solution is leached out of the soil by a solution of lower concentration, for example, rain water or snow melt, then the micelle will expand to its original size (1). The pressure required to suppress this expansion will correspond to the higher potential, ψ_2 . If in addition to the effect of a high concentration solution the soil is consolidated under a pressure, state (3) will be attained. Upon removal of the consolidation pressure the soil will want to attain a potential state lower than that of ψ_3 . If moisture comes in contact with the soil but expansion is suppressed, the swelling pressure will correspond to



- 1 - SOIL IN EQUILIBRIUM AT ELECTRICAL POTENTIAL ψ_1
 2 - SOIL IN EQUILIBRIUM AT HIGHER ELECTRICAL POTENTIAL CORRESPONDING TO POTENTIAL OF SOLUTION CONCENTRATION ψ_2
 3 - SOIL IN EQUILIBRIUM WITH BOTH SALT SOLUTION AND APPLIED PRESSURE ψ_3

FIGURE - 6. ILLUSTRATION OF THE ELECTRICAL POTENTIAL EQUILIBRIUM STAGES OF THE CLAY MICELLE

potential, ψ_3 .

Whether the soil is subjected to the processes described or to desiccation, the net effect is the same. In all cases, the size of the micelles will achieve certain dimensions to be in equilibrium with the prevailing conditions. Within the micellar solution the cations are dispersed. When the micelle size is small the cations are highly concentrated, and the opposite holds when the micelle enlarges. The phenomenon of varying concentrations of the cations in the clay micelles being in equilibrium with the existing external conditions is essentially an osmotic effect. It must be pointed out that the above discussion applies only when the moisture content of the sample is such that double layers exist.

The testing program accented the study of the swelling pressure of the soils under varying conditions. The results presented in TABLE II are plotted in FIG.7. Pertinent findings from these results are as follows:

1. The sodium clay yielded higher swelling pressures than the calcium clay.
2. At high initial moisture contents, the samples tested in lower concentration solution yield higher swelling pressures.

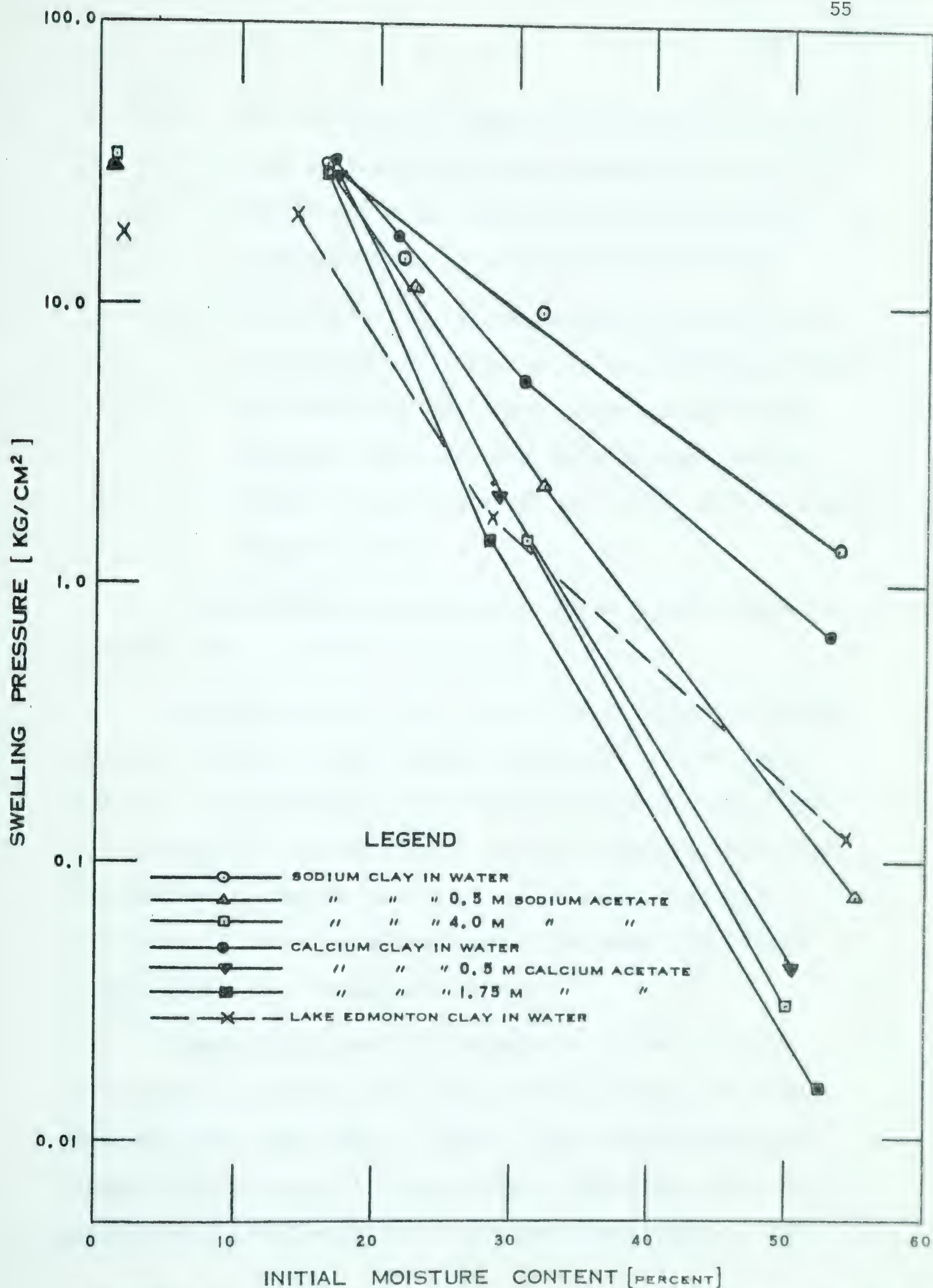


FIGURE 7.- SWELLING PRESSURE VS. INITIAL MOISTURE CONTENT

3. As the initial moisture content is lowered the swelling pressures increase but the differences in the pressures for varying concentration of solutions are reduced.
4. For initial moisture contents close to the shrinkage limit and below the shrinkage limit the swelling pressures obtained are almost the same for both the calcium and sodium samples regardless of the nature of the immersing solution.

From preceding discussions items 1 and 2 were as expected and no further comment is necessary.

With regard to item 3 which deals with the initial moisture content, ideal osmotic behaviour of the clay micelles is increasingly obscured as the moisture content of the samples becomes lower. At low moisture contents the particles come so close to one another that the influence of the attractive forces dominate the effect of the increasing repulsive forces.

Concerning item 4 it should be stated that at low moisture contents the water layers around the clay particles are very thin. Virtual particle to particle contact will prevail if the strongly adsorbed water is considered to act as part of the particle itself. The

adsorbed layer at these low moisture contents is so thin that any increase in concentration of the pore water has no further thinning effect. At this stage the osmotic phenomenon is considered to be no longer applicable. The behaviour of the sample will depend on the interaction of attractive and repulsive forces emanating mainly from particle hydration.

The slightly different behaviour of the Lake Edmonton clay sample can be attributed to two causes:

- (a) The Lake Edmonton clay sample has a lower percentage colloidal sizes than either the sodium or calcium samples.
- (b) The salt content of the Lake Edmonton clay is much higher than the salt content in either the sodium or calcium samples.

A lower percentage of colloidal size particles would yield lower swelling pressures, higher salts in the sample will also yield lower swelling pressures. These two factors combined explain the lower swelling pressures for the Lake Edmonton clay.

The test results indicate that at higher moisture contents the swelling and swelling pressures are caused mainly by the osmotic effect. An attempt to correlate the observed swelling pressure with calculated osmotic

pressure based on the existing equation, (See Chapter V), did not show a close relationship. Bolt (1956) obtained success in this direction but his experiments were performed under ideal conditions as explained in Chapter II.

So far the discussion has centered on tests which have accented the osmotic effect on swelling. Test No. 5 was performed using varsol as the immersing solution for the Lake Edmonton clay. Varsol is essentially a non-polar solution compared with water. Therefore, osmotic pressure was not expected to develop between the soil and the varsol. When this sample was consolidated and re-bounded practically no swelling was observed.

If capillary effect is assumed to contribute to swelling when water is used as a result of the destruction of the capillary menisci between soil particles, then with the use of varsol the sample would be expected to expand approximately proportional to the surface tensile properties of the varsol compared with water. Water has a surface tension of 71.47 dynes/cm at 25°C (Handbook of Physics and Chemistry) while varsol is assumed to have a surface tension of 20 dynes/cm. A definite surface tension of varsol could not be obtained but an approximation was based on the surface tension of kerosene 22.5 to 32.5 dynes/cm and naphthas 19 to 23 dynes/cm at 25°C. (International Critical Tables). The

properties of varsol are assumed to be not too different from kerosene and various naphthas. On this basis the effect of capillarity can be expected to have at least 25% effect using varsol instead of water. But the test with varsol showed practically no swelling. The tests using pulverized sand tested in water also showed no appreciable rebound. The pulverized sand can be considered inert with water as compared with the reaction between water and clay. The majority of water retained in the sand sample is held by capillary tension.

4.7 Rebound Characteristics

The slope of the rebound curve taken from a void ratio versus pressure plot is called the swelling index. Values of the swelling index for the tests were determined between load increments 0.1 and 1.0 tons/ft² and are tabulated in TABLE II. The values obtained are approximately the same for all the tests except for the sand samples and for the test with varsol. The swelling indices for the sand samples and for the clay tested in varsol are essentially zero. These samples exhibited little or no swelling. The narrow range of values for the swelling indices obtained for the other tests indicate that the rebound curves were generally parallel to each other. This same trend was obtained by Hamilton (1961)

for the calcium and natural soils. Hamilton was able to obtain the swelling indices for the sodium sample tested in water and found they were about 4.5 times that for the natural soil and calcium samples.

Rebound curves presented in FIG. 8 show the amount of rebound versus time for the load decrement going from 20 grams on the pan to zero load on the pan. Curves 1 and 2 represent rebound of the calcium samples with 1% and 30.7% initial moisture contents tested in water. The rebound curve for 1% initial moisture content shows that this sample swelled more under the same load decrement. Curves 2 and 3 represent rebound of the calcium samples for approximately the same initial moisture content but curve 2 is for sample tested in water and curve 3 is for sample tested in 0.5M Ca Ac₂. These curves indicate the decrease in amount of swell when a sample is in contact with a salt solution instead of water for the same load decrement, stress history and initial moisture content. Curves 3 and 4 represent rebound curves for calcium and sodium samples with approximately same initial moisture content, stress history and with 0.5M salt solution as the immersing solution. These curves show the higher amount of swell for the sodium sample as compared with the calcium sample. The nature of these curves with respect to amount of swell is in accordance with the

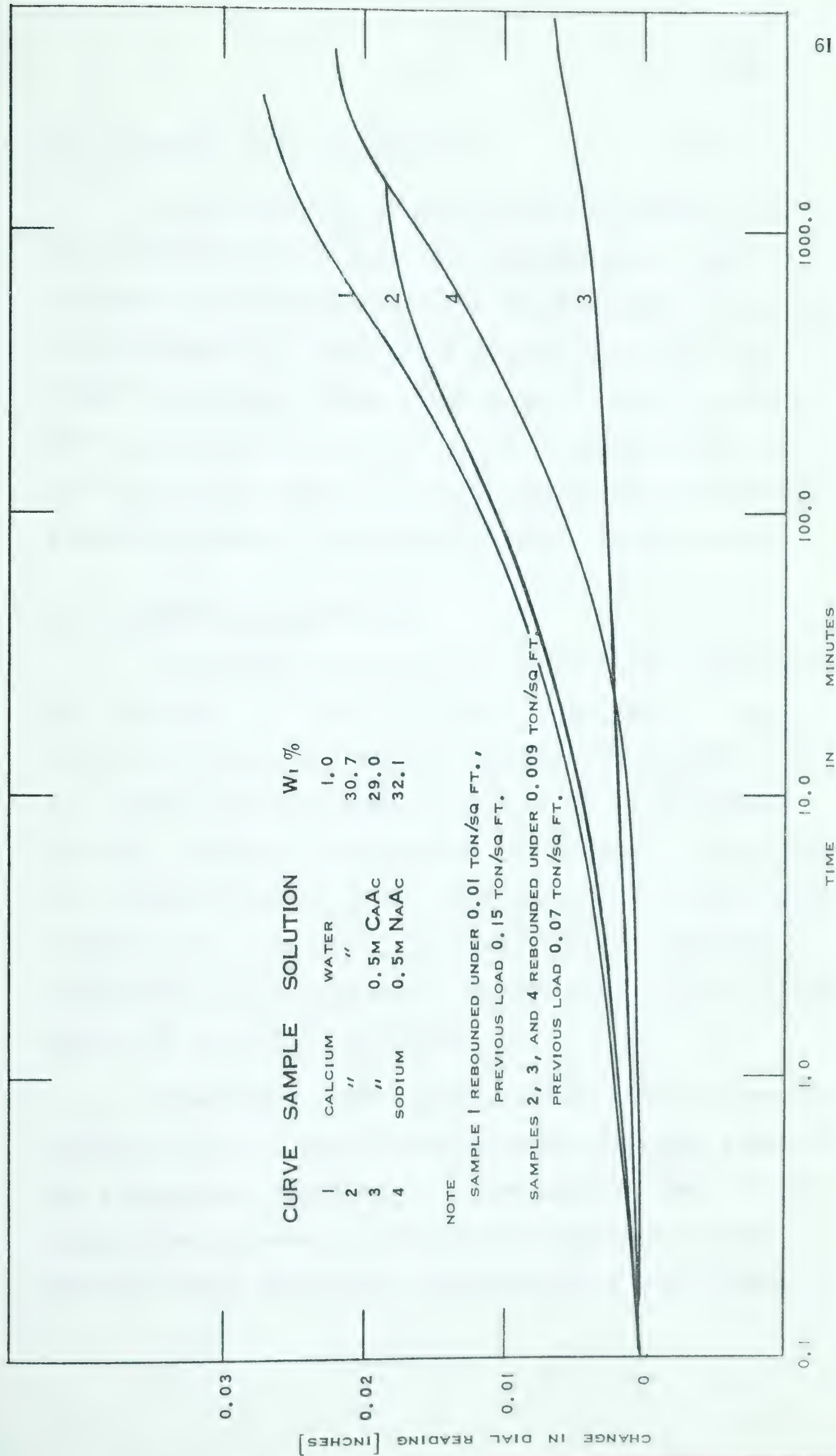


FIGURE 8. - REBOUND VS. TIME [VARYING TEST CONDITIONS]

discussion already presented.

With regard to time for primary rebound it was observed that at the high load decrements the time required was somewhat less than that required at the low load decrements. There was a general trend for equal times for primary rebound with samples tested in water and in salt solutions. It was also observed that the sodium samples tended to take a longer time to complete primary rebound as compared with the calcium samples.

4.8 Degree of Saturation

The initial degree of saturation for the samples are tabulated in TABLE II. These values show a consistent decrease in degree of saturation from 99% at about 50% initial moisture content to 5 or 6% at 1% initial moisture content. This decrease in degree of saturation will always occur in nature when the soil is subjected to desiccation. Consolidation due to positive pressure (overburden) will maintain 100% saturation if the initial degree of saturation was 100%.

Fredlund's (1964) investigation showed that consolidation due to both desiccation and positive pressure was essentially the same up to the plastic limit of the soil. Similar results were obtained because in both types of tests the samples used were in a high degree

of saturation. Below the plastic limit the desiccated samples undergo smaller volume change compared with the water extracted, and from the shrinkage limit no further volume decrease occurs. This is quite different from a positively consolidated sample, for example, the Bearpaw Shale of Western Canada. Hardy (1963) reported that the undisturbed formations (50 feet below surface) have natural moisture contents generally less than 10% which is certainly lower than the shrinkage limit.

It should be pointed out here that the nature of the results obtained from the testing program for samples having low moisture contents cannot be applicable to the Bearpaw Shale when the moisture content is lower than the shrinkage limit but with the sample still saturated, because of the difference in the type of consolidation. In the case of the desiccated samples, when the water is extracted from the samples below the shrinkage limit air voids are created. These voids are theoretically filled again when water comes in contact with the soil, without causing the soil to expand. Really then, the swelling pressures measured in the testing program for samples below the shrinkage limit, represent swelling pressures not at the indicated initial moisture contents but at a moisture content corresponding approximately to the shrinkage limit. This will explain why similar results

are obtained for samples tested at initial moisture contents close to the shrinkage limit and for samples with 1% initial moisture content.

In the case of the Bearpaw Shale, moisture loss below the shrinkage limit was accompanied by volume decrease. This moisture loss was brought about by very high confining pressures which in essence may be considered as dehydration of the particles. Van Olphen (1962) showed that this was possible and he reported squeezing out the second last molecular layer of water from a clay particle with a pressure of 30,000 p.s.i. If the Bearpaw sample in its low moisture state is exposed to the atmosphere, it will adsorb water molecules from the air depending upon the relative humidity, the nature of the adsorbed cations and the mineralogy of the particles. Though the amount of swell resulting from this process (particle hydration) will be small, the swelling pressure will be tremendous. The process of particle hydration will continue, depending on the mineral type and atmospheric conditions up to or beyond the shrinkage limit. If then liquid water comes in contact with the soil greater swelling, but at reduced pressure, will result.

4.9 Summary

In this chapter the test results were presented

and a discussion of the results was made. The tests on the sodium and calcium samples showed that the amount of swell and swelling pressures were greater for the sodium samples compared with the calcium samples. It was also observed that the amount of swell and swelling pressures were higher for samples tested in lower concentration solutions. As the initial moisture content of the samples decreased the differences in amount of swell and swelling pressures were reduced. At initial moisture contents close to the shrinkage limit the results were essentially the same for all samples. The clay sample tested in varsol and the sand samples displayed no appreciable swelling.

Also considered in this chapter was the amount of water imbibed by the samples, rebound characteristics, and the differences in behaviour of desiccated samples as compared with samples consolidated under positive pressures.

CHAPTER V

DISCUSSION ON MECHANISMS OF SWELLING IN CLAY

5.1 Introduction

A mechanism of swelling has been defined as a physico-chemical phenomenon that occurs in the soil causing the soil to swell. The following postulates have been advanced to explain the swelling of clays:

- (a) Hydration of clay particles.
- (b) Hydration of adsorbed cations.
- (c) Attraction of water molecules by the clay particle.
- (d) Capillary effect.
- (e) Osmotic effect between the adsorbed water of the clay particle and the outside solution.
- (f) Osmotic effect between the soil mass and the outside solution.

In this chapter the above postulates will be discussed in the light of the existing literature and on the basis of the test results. The purpose of this discussion is to evaluate the postulates as they apply to soil engineering.

5.2 Hydration of Particles

An attempt to investigate the effects of particle hydration using desiccated samples with initial moisture content of 1 percent was unsuccessful because desiccated samples attain a minimum volume corresponding to that at the shrinkage limit. Below the shrinkage limit desiccated samples undergo moisture content variation at constant volume of the sample. However, on the basis of existing literature Mattson (1932), Katz (1933), Marshall (1949), Barshad (1955), McEwan (1955) and Van Olphen (1962) there is ample evidence to show that expansion of a dehydrated soil mass is first caused by particle hydration. Particle hydration can be assumed to be complete at a relative humidity of close to 100%.

Based on data from observations using the X-ray diffraction technique Barshad (1955) showed that the dehydrated clay particle would adsorb water from the atmosphere. The manner in which the adsorption process occurred was described in Chapter II of this thesis. The molecules of water would first be adsorbed on the external surface of the particle. If the mineral was the expanding lattice type (e.g. montmorillonite) then water molecules would enter between the lattice structure and cause extra volume change. If the mineral was not of the expanding type (e.g. kaolinite) then the amount of water adsorbed

will be much less. Results presented by Barshad (1955) showed that the sodium montmorillonite adsorbed water to the extent of 36.5% moisture content at a relative humidity 95%, at the same condition the sodium kaolinite adsorbed water to the extent of only 2.5% moisture content. Therefore, the degree of moisture adsorbed due to particle hydration depends on the type of minerals involved, as a consequence, the mineralogy of the clay particles governs the amount of expansion a soil exhibits due to particle hydration.

The minimum volume a natural soil can normally attain is the volume at the shrinkage limit unless the soil is highly overconsolidated. If it is assumed for the sake of argument that 12% is a representative value of the moisture content for the shrinkage limit of clays, then if the clay is predominantly of kaolinite, expansion due to particle hydration will not occur because at 95% relative humidity kaolinite exists at only 2.5% moisture content, well below the shrinkage limit. In case the natural soil is of pure sodium montmorillonite, at 95% relative humidity the sample will exist at 36.5% moisture content well above the shrinkage limit (even above the plastic limit) and variations in volume change of the soil can occur due to atmospheric variations. However, if the relative humidity

of the natural atmosphere is assumed to be 40% (as it is in the average case) the moisture content of the sodium montmorillonite will drop to approximately 11.5%, which corresponds with the shrinkage limit and no major volume changes are expected to occur due to variations in atmospheric conditions. In addition, natural soils are seldom 100% sodium montmorillonite, hence, for practical purposes it can be assumed that under normal atmospheric conditions natural soils will not be subject to volume changes due to particle hydration.

Highly overconsolidated soils that are compressed to a volume below the volume corresponding to the shrinkage limit, will adsorb water through the process of particle hydration depending upon the atmospheric conditions and the mineral type of the clay particles.

5.3 Cation Hydration

The test results for the sodium and calcium clays show that the nature of the adsorbed cations does influence the swelling behaviour. The swelling pressures obtained for the sodium clay are shown in FIG. 7 to be consistently greater than those for the calcium clays at moisture contents exceeding 20 percent for the samples tested in water. Below 20 percent moisture content, the calcium clay shows a higher swelling pressure. The samples

at approximately 1 percent initial moisture content did not reveal the results expected because the samples desiccated below the shrinkage limit when immersed again in water will show tendency to swell only after the moisture content at the shrinkage limit has been achieved.

The data presented by Barshad (1955) regarding water adsorption by sodium and calcium montmorillonite from a dehydrated state shows that the calcium soil adsorbs more water than the sodium soil at the same relative humidity (FIG.1). Although no published data exist relating the swelling pressures for these soils at low moisture contents, it is suggested that the calcium sample is more likely to exert the higher pressure at low moisture contents. This is based on the fact that the test results for this thesis show that the swelling pressure curve for the calcium sample tends to be higher than that for the sodium sample, (FIG.7) below 20 percent moisture content. In addition, the adsorption isotherms (FIG. 1) indicate that the calcium sample adsorbs more water at a particular moisture content and hence will be expected to expand more.

The concept of cation hydration on swelling of soils was discussed by Havlicek (1961). He stated that monovalent ions smaller than 1.32\AA radius should behave as hydrophillic (water attracting) particles, while

larger ones should behave as hydrophobic particles and should be regarded as negatively hydrated. This idea was based on tests in which monovalent ions larger than 1.32\AA as the crystalline radius showed smaller radius on the basis of Stokes principle from their speed in water solution in an electric field. The smaller observed radius indicated that the large ions speed up diffusion of water whereas the others slow it down. In addition, Havlicek found that the concentration and density of water in the proximity of the larger ions were less than that in the proximity of the others. On this basis Havlicek concluded that the cations larger than potassium form thinner diffuse layers than lithium and sodium thereby reducing the swelling of soils.

Grim (1958) commented on the influence of the adsorbed cations:

"They serve as a bonding force between particles, and this may determine the available space between particles and the attractive force that growing layers of water must overcome in pushing apart clay particles. They determine the thickness of the oriented water layers also by the manner in which the cations or the cation plus a hydration envelope can fit into the geometry of the water net. The fit or absence of a fit may serve to enhance or retard the formation of oriented water layers. The cation may

determine the actual geometry of the water net, and of very great importance, determine whether the oriented water grades gradually into liquid water or is abruptly separated from liquid water".

According to Grim (1958) with calcium as the adsorbed cation the clay particle in the presence of water will develop oriented water to the thickness of about four molecules of water, beyond this thickness there will be an abrupt change to liquid water. In the case of the sodium cation thick oriented water layers are formed and there is no abrupt change from the oriented water to liquid water.

5.4 Attraction of Water Molecules

The "Soil-Water Potential Theory" was presented as a discussion at the Twelfth Canadian Soil Mechanics Conference held in 1958. According to this theory every soil particle is able to adsorb a certain amount of water on its surface by the molecular attraction and every water molecule is attracted toward the solid surface in the same way. A critical examination of this theory shows that no consideration is given to the nature of adsorbed cations, clay mineral type, or to the presence of a solution other than water. The test results of this thesis show that these effects have important influences on the swelling behaviour of the soil. In the circumstances the "Soil Water Potential Theory" is too limited to fully explain

the behaviour of soils.

5.5 Capillary Effect

When a soil sample is desiccated capillary menisci are developed in the soil pores. Also, when a consolidated soil sample is unloaded the tendency for the mass to expand creates capillary menisci in the soil pores. In both circumstances a destruction of the capillary menisci reduces the tension in the fluid pulling the soil particles together and the soil mass expands. One method of determining the capillary pressure (Skempton 1961) is to find the no-volume-change pressure. The results of the tests (TABLE II) show that by varying the concentration of the immersing solution different pressures are obtained for samples having approximately the same initial characteristics, for example, tests Nos. 6 and 14. These differences indicate the effect of the surface action of the clay particles in an environment of a polar substance such as water and the salt solutions.

To eliminate the effect of surface action varsol was used instead of water, (test No. 5). The sample was consolidated to 18.4 kg/cm^2 but with unloading very little rebound occurred. The validity of this test as a measure of the capillarity effect on swelling is questionable on the basis that the varsol might have changed the properties

of the clay. The results of the pulverized sand sample (test No. 30), in which surface action was assumed negligible, also showed very little rebound on release of load. The sand was pulverized to yield a grain size distribution approximately comparable to the clay sample and as such, the pore sizes would be close to those in the clay sample and would therefore generate a similar capillary tension using water. The results of this test may be questioned on the basis of the difference in shape of the solid particles as compared with the clay particles.

In the circumstances, the attempt to isolate the effect of capillarity on swelling was not achieved. It was, however, demonstrated that in a clay where particle surface action is significant all the swelling observed is not due to the release of capillary stresses.

5.6 Osmotic Effect Between the Adsorbed Water Phase and the Outside Solution

The concept of the osmotic effect on swelling is based on the interaction of the concentration of the ions within the adsorbed aqueous phase of the clay micelles and the prevailing conditions in the outside solution. The theoretical implications of this concept as applied to soils can be extremely involved as discussed by C.E. Marshall (1964). For the purposes of the soils engineer it will be worthwhile to at least have a basic

knowledge of this concept in order to take it into consideration where applicable in his design work.

The discussion of the test results in Chapter IV, made on a qualitative basis, gave a good indication that the phenomenon of osmosis could be applied to explain the swelling behaviour of clays provided the moisture content was high enough to allow formation of double layers. At a moisture content of approximately 50% it was observed that the sodium sample tested in water yielded the higher swelling pressure when compared with the calcium sample tested in water. This could be explained by the osmotic theory since the osmotic pressure is a colligative property and depends on the number of ions in the adsorbed water phase. The sodium and calcium samples had about the same cation exchange capacity (refer to TABLE I) and therefore since sodium is univalent and calcium bivalent, there would be twice as many sodium ions as calcium ions in the adsorbed phase resulting in a higher osmotic pressure for the sodium sample. But if potassium ions were used, potassium being univalent, it would be found that the potassium sample would not yield a similar swelling pressure as the sodium sample. This statement is based on the nature of the results obtained by A.B. Hamilton (1961). This would point out one limitation of applying the over

simplified osmotic pressure equation to soils:

$$P_s = R T C \dots\dots\dots 2$$

Ruiz (1963) modified this equation to:

$$P_s = R T \frac{''T''}{i Va} f$$

where ''T'' is the cation exchange capacity,

i is the moisture content of the sample ,

Va is the valence of the cations ,

f is an activity constant.

Ruiz replaced C, the concentration of the ions in the adsorbed phase in moles per litre, by using the cation exchange capacity, moisture content and valence of the cations as follows:

Cation exchange capacity per 100 gms dry soil =

$$\frac{''T''}{1000 \times Va} \text{ moles per 100 gms of soil.}$$

This number of moles is uniformly dispersed in

''i'' grams of water contained in 100 gms of soil,

∴ Average concentration of cations(moles/litre),

$$\begin{aligned} C &= \frac{''T''}{1000 Va} \times \frac{1000}{i} \\ &= \frac{''T''}{i Va} \end{aligned}$$

The activity value (f) could be obtained by dividing the observed swelling pressure by the osmotic pressure calculated from:

$$P (\text{ideal}) = R T \frac{''T''}{i Va}$$

This procedure would be acceptable provided that the difference between the observed and calculated osmotic pressures was due to the degree of dissociation of the cations in the aqueous phase. Some calculated values are presented in TABLE VI.

TABLE VI : Osmotic Pressure and Activity Coefficient

Test No.	Sample	Wi %	P ideal kg/cm ²	P(Observed) kg/cm ²	f
6	Calcium	52.9	7.48	0.65	0.087
7	Calcium	30.7	12.9	5.3	0.412
9	Calcium	16.7	23.6	33.0	1.4
18	Sodium	53.6	14.7	1.3	0.088
19	Sodium	31.9	24.8	9.2	0.372
21	Sodium	16.7	47.3	31.0	0.65

The values tabulated in TABLE VI show that the laboratory swelling pressures observed for the sodium clay are about twice that for the calcium clay even at initial moisture contents of 30%. At 16% moisture content the picture is completely changed, the calcium sample shows a slightly higher swelling pressure than the sodium clay. According to the equations for calculating the osmotic pressure as proposed by Ruiz, the ideal osmotic pressure

for the sodium clay will always be twice as large as that for the calcium clay; any deviation from this behaviour Ruiz simply attributes to activity and these calculated values are tabulated in TABLE VI.

The simplified method proposed by Ruiz for accounting for the apparent discrepancy between observed and calculated osmotic pressures disregards natural behaviour of soils. As pointed out by Ruiz no attention is given to the effect on the clay particle behaviour when different cations, but of the same valence, are adsorbed; no consideration is given to the particle to particle interaction which introduces friction effects nor to the fact that the adsorbed water layer at low moisture contents no longer behaves as liquid water so as to act as a true dispersing medium for the cations, resulting in the absence of any osmotic phenomenon. In addition, Ruiz in his simplified method does not take into account the effect of attractive forces which possibly come into play when the moisture content is lowered. However, Ruiz has done a great deal to accentuate the importance of the osmotic theory for application to soil engineering. His simplified method was merely to present a workable hypothesis for practical applications. In so doing one must be made aware of the dangers of oversimplifying the complex osmotic theory

as applied to natural soils.

An important observation arising from the research for this thesis was that regardless of the nature of the immersing solution, at moisture contents slightly above the shrinkage limit, the swelling pressures were approximately the same for both the sodium and calcium samples. This condition gives a clear indication that while at high moisture contents there is evidence of an osmotic behaviour of the soil, this ideal osmotic behaviour is gradually diminished as the moisture content of the soil decreases. Since the existing osmotic equation does not account for the differences in behaviour of natural soil at varying moisture contents there is reason to visualize why attempts have been futile in relating observed swelling pressures with calculated osmotic pressures based on the existing equations. Limited success has been achieved so far in relating these calculated and observed values for special conditions as mentioned in CHAPTER II.

5.7 Osmotic Effect Between the Soil Mass and the External Solution

This postulate is based on the assumption that the cations and anions existing in the free pore water are a source of osmotic pressure if a solution entering from an external source is at a lesser salt concentration than

the pore water. In this case the "membrane" separating the solutions is assumed to be the soil mass itself with its capillary network, that is the external solution of low concentration can move into a soil mass but the pore water originally present cannot move out.

Tests Nos. 30 and 31 using the pulverized sand were directly concerned with this aspect of swelling. Test No. 30 was performed with the sand immersed in water, and no tendency for swelling was observed. For test No. 31 1.231 grams of sodium acetate salt was powdered and mixed with 50 grams of the sand in order to achieve a salt solution of approximately 0.5M concentration in the pore water based on the results of test No. 30. Sample behaviour during this test showed that under the minimum load 0.009 kg/cm^2 temporary swelling occurred in the initial stages but to a rather small extent. With time the sample decreased in volume under the same minimum load. During the test, checks were made on the conductivity of the immersing water. It was found that the conductivity increased with time indicating that the salt originally in the sample was migrating out into the immersing solution.

The nature of the results of these tests indicate that an osmotic swelling develops as a result of the difference in concentration between the soil pore water

and a less concentrated external solution. This osmotic swelling is temporary since the soil mass does not effectively prevent migration of ionic particles in the free pore water out of the soil mass. This condition is unlike the case of the ionic particles in the water phase of the clay micelle where the ions are held in check by the electrostatic potential of the clay particle.

5.8 Elastic Rebound

Elastic rebound normally occurs immediately after the soil is unloaded. Sometimes, for example, in the case of a mica type material, loading causes not only elastic deformation but also bending of particles. When the load is removed immediate response in volume change may be retarded due to frictional forces preventing the bent particles from straightening out. However, elastic rebound is usually small compared with total swelling in clays, and does not normally constitute a critical problem.

Elastic rebound as considered above refers to expansion due to elastic restitution of the soil particles as well as expansion of air pockets. This is the concept of elasticity as used in this thesis. Sometimes elastic rebound refers to the total expansion a soil undergoes upon release of loading, in this case, meaning expansion due to imbibition of water over and above that caused by

elastic restitution of the soil particles and expansion of the entrapped air.

5.9 Concept of Neutral and Effective Stresses as Applied to Soil Volume Changes

The term effective stress is defined as the stress controlling changes in volume or strength of the soil, (Skempton, 1960). The neutral stress is the stress in the fluid phase of the soil mass, this is also referred to as the pore pressure.

When a mass of porous material is subjected to a small increase in pressure from p' to $p' + \Delta p'$ under the condition of zero pore pressure the volume changes from V to $V + \Delta V$ where ΔV is negative, the compressibility, C , of the material for this particular pressure increment is defined by the equation:

$$- \frac{\Delta V}{V} = C \Delta p'$$

Compressibility is not a constant, but decreases with increasing pressure and eventually, under a pressure sufficiently high to eliminate the voids, C will fall to a value C_s , where C_s is the compressibility of the soil solids.

Using the relationship,

$$\Delta p' = \Delta p - \Delta u$$

where Δp is a change in total stress and Δu is a change in pore pressure,

the expression, $-\frac{\Delta V}{V} = C (\Delta p - \Delta u)$, is obtained which is the expression suggested by Skempton (1960) to relate volume change with changes in effective stress. This equation implies that a volume change must occur if the effective stress is changed for a soil having a compressibility, C . On this basis the consolidation and rebound of a soil as a result of changes in effective pressure is said to occur due to the compressibility and expansibility of the soil mass. The total rebound of the soil mass is attributed to a so-called elastic rebound occurring in three stages. The first stage is an immediate rebound on removal of load, the second stage is the primary rebound which is attributed to expansion at a slower rate as time is required for liquid to enter the soil mass; and the third stage is a secondary rebound due to elasto-plastic deformation.

However, the results from the testing program, plotted in FIG. 9 show that for any given void ratio, above approximately 0.65, a soil (calcium or sodium clay) can show increases in effective stress by testing the soil in solution environment of lesser concentration. That is, since constant volume tests are involved observed changes in swelling pressure must lead to different effective stresses. For Skempton's (1960) theory to be valid it must, therefore, be argued that the nature of the testing

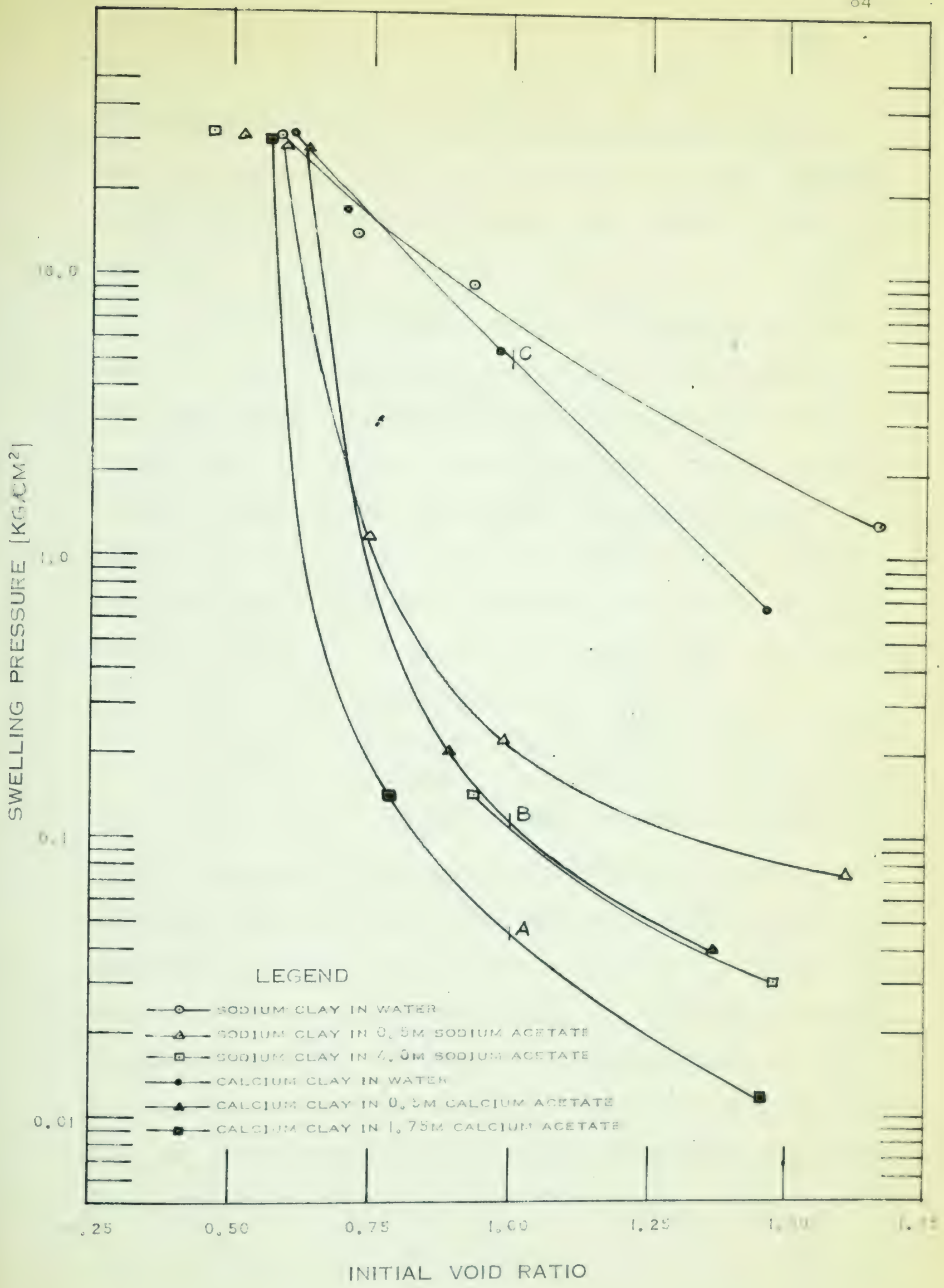


FIGURE 9 SWELLING PRESSURE VERSUS INITIAL VOID RATIO

solution either changes the elastic properties of the soil or causes the effective stresses to change. Skempton (1960) did not appear to consider this aspect of soil behaviour.

An answer to this problem is suggested by the physico-chemical concepts. According to this concept a soil can exhibit changes in behaviour because of changes in the surface effects of soil particles. These surface effects depend on factors such as adsorbed cations, mineralogy of particles, size of particles and nature of the pore water. Consider for example the curves for the calcium samples at an initial void ratio 1.00, the apparent effective stresses obtained were as follows:

Case A, 4.8 kg/cm^2 , sample tested in water

Case B, 1.1 kg/cm^2 , sample tested in 0.5M Ca Ac_2

Case C, 0.4 kg/cm^2 , sample tested in 1.75M Ca Ac_2

These differences in apparent effective stresses can be attributed to the variation in the osmotic pressures developed (See Section 4.6). Since the initial void ratio is constant, it can be reasoned that the capillary pressures developed would be the same for the three cases, A, B, and C as a result of desiccation. This capillary pressure will be of the same magnitude as the negative pore pressure. This pore pressure can be identified as,

$$u = u_A = u_B = u_C$$

Beside the pore pressure in the soil, there exists the effect of surface action in the form of a net interaction of the attractive and repulsive forces called the $(R - A)$ forces which act in the same manner as a positive pore pressure in that they tend to push particles apart. These forces can be identified as,

$$(R - A)_A, (R - A)_B, (R - A)_C$$

The attractive forces, A , depend only on particle separation and is relatively insensitive to system characteristics (Lambe 1960). Since the void ratios are constant, hence it may be considered that,

$$A_A = A_B = A_C$$

The repulsive forces depend on particle separation and also on the system characteristics. Therefore, R can be separated into two components R_d , distance dependent and R_o , system characteristics dependent. Since the void ratios are constant, particle distances may be considered to be constant, and hence,

$$R_{dA} = R_{dB} = R_{dC}$$

would appear to be a reasonable assumption.

However, R_{oA} , R_{oB} and R_{oC} will differ because the concentration differences of the immersing solution have an effect on the thickness of the adsorbed water

layers and for this reason it can be classified as an osmotic pressure.

Adhering to the concept that effective stresses remain constant with constant volume then the u , R_d and A forces will be a direct function of effective stress and they remain the same for the desiccated sample, A , B and C at the same initial void ratio. However, the force component, R_o is such that it can be made to fluctuate by inducing base exchange of the adsorbed ions or varying the pore water salt concentration. It is essentially an osmotic pressure, as defined and can be considered as a force superimposed on the u , R_d and A forces.

During the constant volume stage of loading any tendency for the soil to expand is suppressed by increasing the load on the sample. The force components trying to bring about expansion will be balanced by the applied load which measures the swelling pressure at equilibrium conditions. An equation can then be written:

$$P_s = u + (R_d + R_o - A)$$

$$\text{for case A, } 4.8 \text{ kg/cm}^2 = u_A + (R_{dA} + R_{oA} - A_A)$$

$$\text{for case C, } 0.4 \text{ kg/cm}^2 = u_C + (R_{dC} + R_{oC} - A_C)$$

$$\text{subtracting, } 4.4 \text{ kg/cm}^2 = R_{oA} - R_{oC}$$

In other words, the difference in swelling pressure

between cases A and C is due to a development in osmotic pressure ($R_{oA} - R_{oC}$), of magnitude 4.4 kg/cm^2 . The magnitude of this difference as compared to the swelling pressure for case C indicates that the osmotic effect seems to be the predominant mechanism contributing to swelling at the void ratio of 1.0 for the sample tested in water. Similar reasoning may be applied to other cases for the range of void ratios shown in FIG.9. The vertical displacement of the curves gives an indication of the effect of osmosis for different soil types and adsorbed cations. However, it must be borne in mind that the force components u , R_d and A not only increases as the void ratio increases, but they may vary individually. For example, at low void ratios A may tend to become greater than R_d . In addition, from the definition of osmotic pressure as used in this thesis R_o will tend to disappear as the initial soil moisture content decreases, and the effect of particle hydration forces will commence.

On the basis of the foregoing considerations it is evident that the effects of osmosis, capillary action, forces of attraction and particle hydration forces may all occur simultaneously in any soil sample. Swelling contributed by osmosis gradually gives way to swelling by particle hydration as the moisture content decreases while the capillary tensile stresses increase in magnitude

with decreasing void ratios. On the basis of the curves in FIG. 9, it is suggested that the upper limit for the osmotic effect be taken as the void ratio at which the curves for different immersing solutions tend to converge on a semi-logarithmic plot. This is based on the premise that as the curves converge, the difference in swelling pressure for tests in solutions of varying concentrations become smaller. This indicates a reduction in the effect of the R_o , or osmotic forces. For the calcium sample this upper limit will correspond to a void ratio of 0.65 or a moisture content of about 20 percent (See FIG.10). For the sodium sample the upper limit can be taken at a void ratio of 0.6 or at a moisture content of approximately 17 percent.

5.10 Other Factors Affecting Swelling of a Clay

From the foregoing discussions it has been shown that particle hydration and osmosis between the adsorbed water phase and external solution were the most significant causes of swelling. To a much lesser extent swelling is due to elastic rebound, capillarity, and osmosis between the soil mass and external solution. It has been pointed out how certain factors, such as, the nature of adsorbed cations, mineralogy of the soil particles, the nature of the external solution, and the existing soil moisture condition can influence the swelling behaviour of a soil.

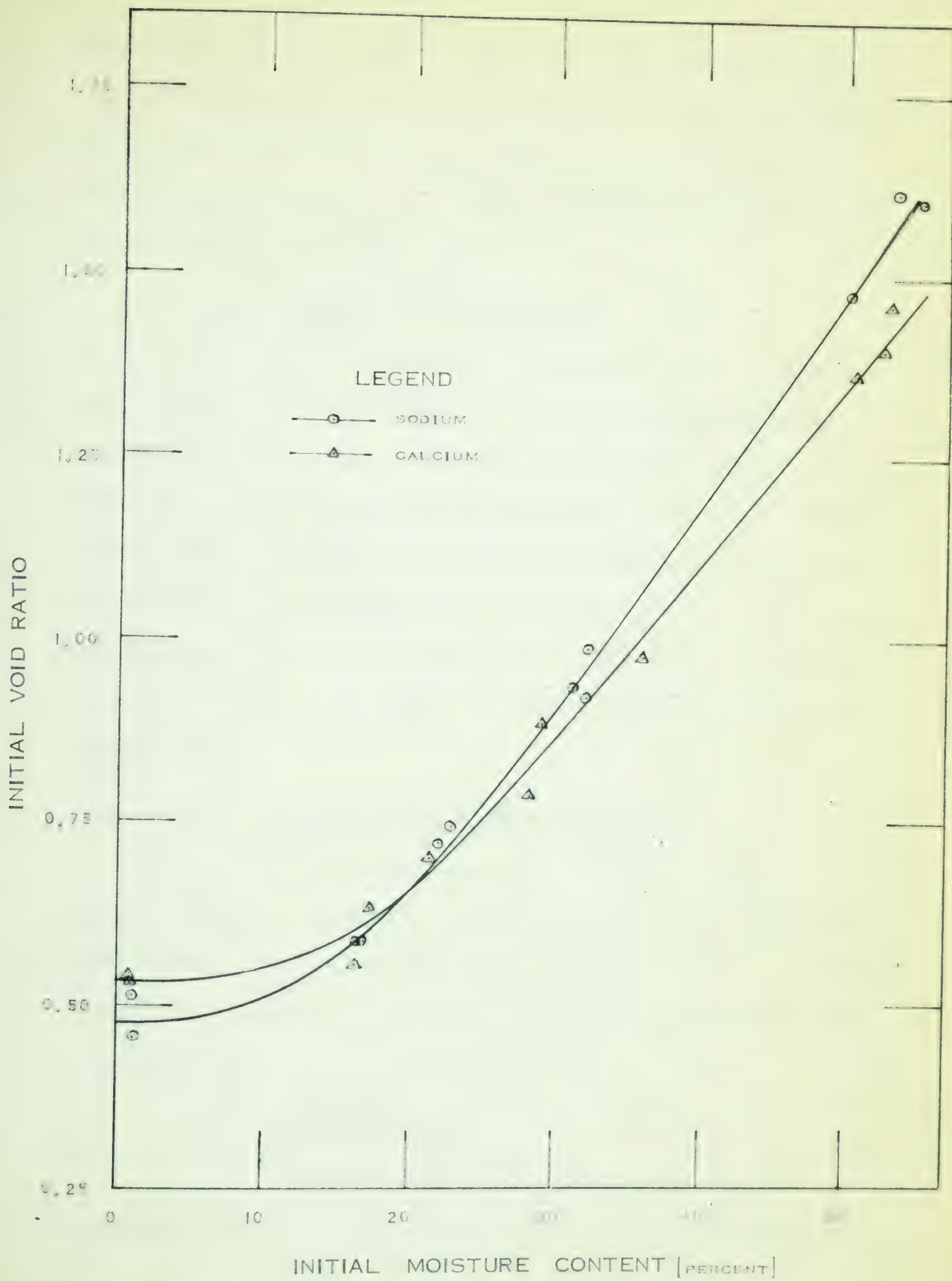


FIGURE 10 INITIAL MOISTURE CONTENT VERSUS
INITIAL VOID RATIO

There are several other factors that influence the swelling behaviour and they are discussed in detail by Seed et al (1962b) and Ladd (1960). These include effects of grain sizes, presence of cementing agents, entrapped air, stress history and time for swelling.

(a) Effect of grain sizes

The presence of clay colloids in the soil mass is the main contributor to water adsorption and desorption. This is because of the larger surface area available and facilitates adsorption of more water molecules. Larger particle sizes have smaller surface area and the amount of water adsorbed will be much less. In addition, the presence of sand and silt particles do not behave as clay particles in that they do not contribute to osmotic imbibition.

(b) Presence of cementing agent

Cementing agents in a soil acts as a glue binding particles together when the sample is compressed. Upon release of load after compression the bound particles cannot partake in soil expansion as would otherwise have occurred.

(c) Entrapped air

When a sample with entrapped air is compressed the air voids will decrease with pressure and

the air may even go into solution. Upon release of load the compressed air will expand. Air expansion from a liquid phase to air phase is much greater than expansion of water or soil particles under the same load decrement. Hence the presence of air in a soil may give rise to much greater volume changes than if the sample was strictly a "soil-water system".

(d) Stress history

Stress history appears to have a significant influence on the swelling behaviour of a soil. It has been observed that with a greater maximum load the soil displays a lower amount of swell when unloaded. It has also been observed that the rate of load decrease from maximum also contributes to the amount of swell. This was observed from test No. 16, calcium sample tested in 1.75M calcium acetate. After consolidating under maximum load, the total load was removed and rebound was allowed. The percent volume change obtained by this method was greater than expected, based on other results obtained using the standard load decrement method.

(e) Time for Swelling

Observations during the testing program indicated that the amount of swelling by a soil depended upon the time as shown by the rebound curves, FIG. 8. Time was required to allow water to migrate into the

sample. The rate of water migration into the sample decreased with time under the same load, yielding a curve analogous to the wellknown consolidation-time curve. It would appear from the secondary swelling branch that a sample could continue to expand to infinite time under a load.

5.11 Summary

In this chapter the various postulates for explaining the swelling of soils were considered with a view to determining the degree of influence of each. Particle hydration appeared to be the main cause of water adsorption in a soil when the moisture content was below the shrinkage limit. As a result, unless the soil was positively compressed to achieve such low moisture conditions, water adsorption due to particle hydration would not contribute to normal soil swelling.

The main cause of swelling in clay soils under natural soil moisture conditions slightly below or above the plastic limit, was considered to be osmosis between the adsorbed water phase of the clay micelle and the external solution. This could not be proved mathematically because the existing equations were not adequate, but the nature of the results from the testing program provided enough evidence to substantiate the assumption. The

capillary effect, elastic rebound and osmosis between the soil mass and external solution were found to be other causes of swelling but generally contributed to the swelling pressure to a minor degree.

CHAPTER VI

CONCLUSIONS

The basic force that causes a clay soil to adsorb water and expand is considered by the soil scientists to be the same regardless of the moisture contents of the soil. From the engineering point of view it is convenient to consider the manner in which the water molecules are adsorbed at varying moisture contents of the soil and to study the behaviour of the soil at the various moisture conditions.

Various mechanisms for water adsorption with consequent swelling of the soil have been postulated and these include particle hydration, cation hydration, attraction of water molecules by the clay particle, capillary effect, osmosis between the adsorbed water of the clay particle and the pore water, and osmosis between the pore water in the soil mass and an external solution of lesser concentration. Some of these mechanisms as postulated occur simultaneously in the soil mass but it is of importance to stipulate the mechanism that predominates at the various soil moisture conditions. The following

conclusions are made on the basis of the investigation:

(a) Particle Hydration

Particle hydration is the mechanism of water adsorption by the soil particles when the soil moisture is so low that the soil adsorbs water molecules from a humid atmosphere. For practical purposes particle hydration can be assumed complete at a relative humidity of close to 100 percent, though there are slight anomalies to this assumption. For most soil engineering purposes the soil condition is such that particle hydration does not appear to contribute to soil swelling. An exception to this is the case where the soil is heavily consolidated to so low a moisture content that particle hydration can contribute to the development of swelling pressures.

(b) Cation Hydration

The specific phenomenon of cation hydration on soil swelling could not be ascertained. However, the nature of the adsorbed cations was found from the tests on the sodium and calcium soils to have significant influence on the swelling behaviour. The adsorbed cations influence soil expansion in more than one way. If the cations are large their movement tends to disrupt the adsorbed water layers resulting in a thinner adsorbed layer and less swelling. If the cations are of higher valence they tend to be more strongly attracted to the soil particles

resulting in thinner adsorbed layers and hence, less swelling. Some cations, for instance the potassium ion form bonds between mineral particles resulting in lower expansion of the mass.

(c) Water Attraction by the Soil Particles

Basically this method of explaining water adsorption is similar to the concepts of the soil scientists. However, the theory does not explain the behaviour of the soil when a solution other than water is in contact with the soil.

(d) Capillary Effect

An attempt to isolate the effect of capillary action on swelling was not achieved. However, in the case of the Lake Edmonton clay in which surface action is significant, it was found that for moisture conditions close to that normally encountered in practice, the capillary effect appears to contribute very little in the total swelling observed.

(e) Osmosis Between the Adsorbed Water Phase and the Pore Water Solution

The nature of the test results indicated that osmosis between the adsorbed water phase and the pore water solution was the main cause of swelling in the Lake Edmonton clay which has a high surface action, at moisture

contents slightly below, and above the plastic limit. A correlation between the observed swelling pressures and calculated values could not be obtained because of the inadequacy of the existing osmotic pressure equations. The test results showed that at initial moisture contents between the plastic and liquid limits of the soil, estimates of swelling pressure were in closer agreement with the osmotic theory but as the moisture content was decreased, gradual departure from true osmotic behaviour was observed. At moisture contents approaching the shrinkage limit it appeared that the osmotic phenomenon was no longer applicable.

(f) Osmosis Between the Soil Mass and the External Solution

The test results showed that swelling from this cause did occur but the amount of swelling was small and occurred only temporarily. The soil mass in its capacity as a semi-permeable membrane did not prevent ions from migrating out of the soil into the external solution, as a result, the swelling initially developed diminished with time under the same load.

(g) Elastic Rebound

Elastic rebound is sometimes referred to as the total rebound the soil exhibits when unloaded, regardless of the fundamental mechanisms that contributed

to the soil expansion. In this thesis elastic rebound was considered to be due to elastic restitution of the soil particles and expansion of entrapped air. Expansion from this cause was expected to be small and would normally occur almost immediately upon release of load. It was not considered a critical problem of swelling in the Lake Edmonton clay sample.

CHAPTER VII

RECOMMENDATIONS

From the research for this thesis it appears that a theoretical answer to fully explain the complex swelling behaviour of soils will not be forthcoming in the near future. Even so, if such an answer should arise it is doubtful whether it could easily be applied to normal practice. On this basis, future work should be directed with a view that the ultimate purpose of the research would be to provide data which could be used to predict the behaviour of more than one type of soil. The current simple classification tests do not provide an adequate answer.

It has been pointed out how factors such as stress history, structure and fabric of the soil, and the presence of salts can affect the soil behaviour. It is therefore recommended as a starting point and for a continuation of the current work, that the effect of stress history on soil swelling be considered. The test series should not necessarily be done on Lake Edmonton clay sample but on artificially constituted specimens to contain varying amounts of different clay minerals and in turn varying the adsorbed ions, moisture content, and salt content. These variants

should be so chosen that with minimum extrapolation the behaviour of any other soil within the range of the varied constituents could be predicted.

LIST OF REFERENCES

REFERENCES

- Barshad, I., "Adsorptive and Swelling Properties of Clay-Water System" Bulletin 169, Division of Mines, State of California, 1955.
- Bjerrum, L., et al, "Some Norwegian Studies and Experiences with Swelling Materials in Rock Gorges" Norwegian Geotechnique No.57, 1964.
- Bolt, G.H., "Physico-Chemical Analysis of the Compressibility of Pure Clay" Geotechnique, Vol.6, No.2, p. 86, 1956.
- Brooker, E.W., "The Influence of Stress History on Certain Properties of Remoulded Cohesive Soils" Unpublished Ph.D. Thesis, University of Illinois, 1964.
- Clisby, M.B., "An Investigation of the Volumetric Fluctuations of Active Clay Soils" Unpublished Ph.D. Thesis, University of Texas, 1962. Geodex 1964-4 (Univ. Micr.Services 62-4830).
- Collins, L.E., "Some Observations on the Movement of Buildings on Expansive Soils Vereeniging and Odendaalrus". Symposium on Expansive Clays, South African Institute of Civil Engineers, 1957-58.

- Dahlman, A.E., "The Influence of Strain on the Shear Strength Parameters of a Highly Plastic Remoulded Homionic Clay Soil". Unpublished M.Sc. Thesis, University of Alberta, 1965.
- Falconer, J.G. and Mattson, S., "The Laws of Colloidal Behaviour. XIII Osmotic Imbibition". Soil Science Vol. XXXVI, 1933.
- Fredlund, D.G., "Comparison of Soil Suction and One-Dimensional Consolidation Characteristics of a Highly Plastic Clay". Unpublished M.Sc. Thesis, University of Alberta, 1964.
- Grim, R.E., "Organization of Water on Clay Mineral Surfaces and Its Implications for the Properties of Clay-Water Systems". Highway Research Board, Special Report No. 40, January 1958.
- Hamilton, A.B., "A Further Study of Cation Effects on the Physical Properties of Clays". Unpublished M.Sc. Thesis, University of Alberta, 1961.
- Hardy, R.M., "The Peace River Bridge - A Failure in Soft Shales". Highway Research Record, No. 17, 1963.
- Hardy, R.M., "Identification and Performance of Swelling Soil Types". Paper presented at 18th Canadian Soil Mechanics Conference, Toronto, Ontario, December, 4, 1964.
- Hardy, R.M. and Dyregrove, A.O., "Practical Experience with Highly Swelling Soil Types". Paper presented to Prairie Regional Soils Conference, Sept. 1962.

- Havlicek, J. and Kazda, J., "Soil Properties in Relation to Hydration of Exchangeable Ions." Proc. Fifth International Conference on Soil Mechanics and Foundation Engineering, 1961.
- Hiller, L.A. and Herber, R.H., "Principles of Chemistry." McGraw-Hill Book Company, Inc. Toronto, 1960.
- Holtz, W.G. and Gibbs, H.J., "Engineering Properties of Expansive Clays" A.S.C.E. Transactions, Paper No. 2814, 1956.
- Katz, J.R., "The Laws of Swelling" Trans. Faraday Society Vol.29, 1933.
- Ladd, C.C., "Mechanisms of Swelling by Compacted Clay" Highway Research Board, Bulletin 245, 1960.
- Locker, J.G., "The Influence of Pore Water Salt Content on the Shear Strength of Remoulded Homionic Clay." Unpublished M.Sc. Thesis, University of Alberta, 1963.
- MacEwan, D.M.C., "Interlamellar Sorption by Clay Minerals" Bulletin 169, Division of Mines, State of California, 1955.
- Maron, S.H. and Prutton, C.F., "Principles of Physical Chemistry" The MacMillan Company, New York, 1958.
- Marshall, C.E., "The Colloid Chemistry of the Silicate Minerals" Academic Press Inc., Publishers, New York N.Y., 1949.

- Lambe, T.W., "A Mechanistic Picture of Shear Strength in Clay". A.S.C.E. Research Conference in Shear Strength of Cohesive Soils, Colorado, 1960.
- Mattson, S., "The Laws of Soil Colloidal Behaviour :VIII Forms and Functions of water". Soil Science V.33, 1932.
- Mattson, S. (1933). See Falconer, J.G. and Mattson, S.
- Sebastyan, G.Y., Written discussion on "The Mechanism of Volume Change in Clays" by B.P. Warkentin. Twelfth Canadian Soil Mechanics Conference, 1959. N.R.C. Technical Memorandum No. 59.
- Seed, H.B., Woodward, R.J., and Lundgren, R., "Prediction of Swelling Potential for Compacted Clays". Proc. A.S.C.E., June 1962 (a).
- Seed, H.B. Mitchell, J.K., and Chan, C.K., "Studies of Swell and Swell Pressure Characteristics of Compacted Clays". Highway Research Board. Bulletin 313, 1962 (b).
- Skempton, A.W., "Colloidal Activity of Clays". Proc. Third Int. Conference on Soil Mechanics and Foundation Engineering, 1953.
- Skempton, A.W., "Horizontal Stresses in Over Consolidated Eocene Clay". Proc. Fifth Int. Conference on Soil Mechanics and Foundation Engineering, 1961.
- Skempton, A.W., "Effective Stress in Soils, Concrete and Rocks". Conference on Pore Pressure and Suction in Soils, London, 1960.

- Streeter, V.L., "Fluid Mechanics" McGraw Hill Book Co., Toronto, 1958.
- Terzaghi, K. Written discussion on "Physico-Chemical Analysis of the Compressibility of Pure Clays" by G.H. Bolt. Geotechnique, Vol. VI, p. 191, 1956.
- Thomson, P.A., "The effects of Exchangeable Cations on Some Properties of Clay Shales". Unpublished M.Sc. Thesis, University of Alberta, 1960.
- Thomson, S., "Effects of Salt Content and Adsorbed Cations in the Shear Strength of a Remoulded Highly Plastic Clay Soil". Unpublished Ph.D. Thesis, University of Alberta, 1963.
- Van Olphen, H., "Compaction of Clay Sediments in the Range of Molecular Particle Distances". Proc. 11th Clay Minerals Conference, Pergamon Press Oxford, 1962.
- Warkentin, B.P., Bolt, C.H., and Miller, R.D., "Swelling Pressure of Montmorillonite". Soil Science Society of America, Proc. 21, 1957.
- Yong, R., Taylor, L.O. and Warkentin, B.P., "Swelling Pressures of Sodium Montmorillonite at Depressed Temperature". McGill University, Soil Mechanics Series No. 4, 1962.

APPENDIX A

VOID RATIO
VERSUS
LOG OF PRESSURE
PLOTS

FIGURE 1A
REM, LAKEEDM. CLAY
SOLN - WATER

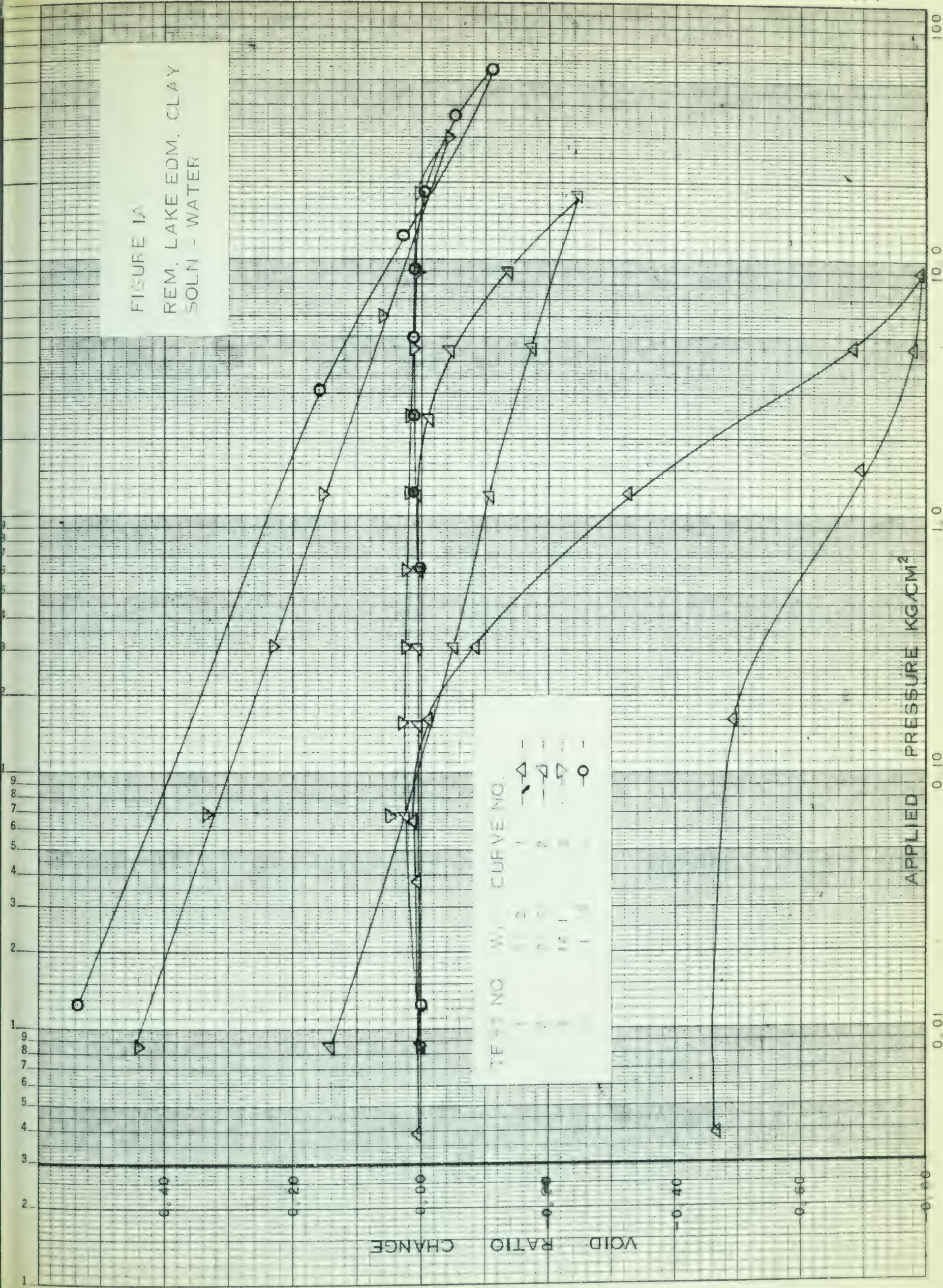


FIGURE 2A
REM LAKE EOM CLAY
COLN - VARSOL

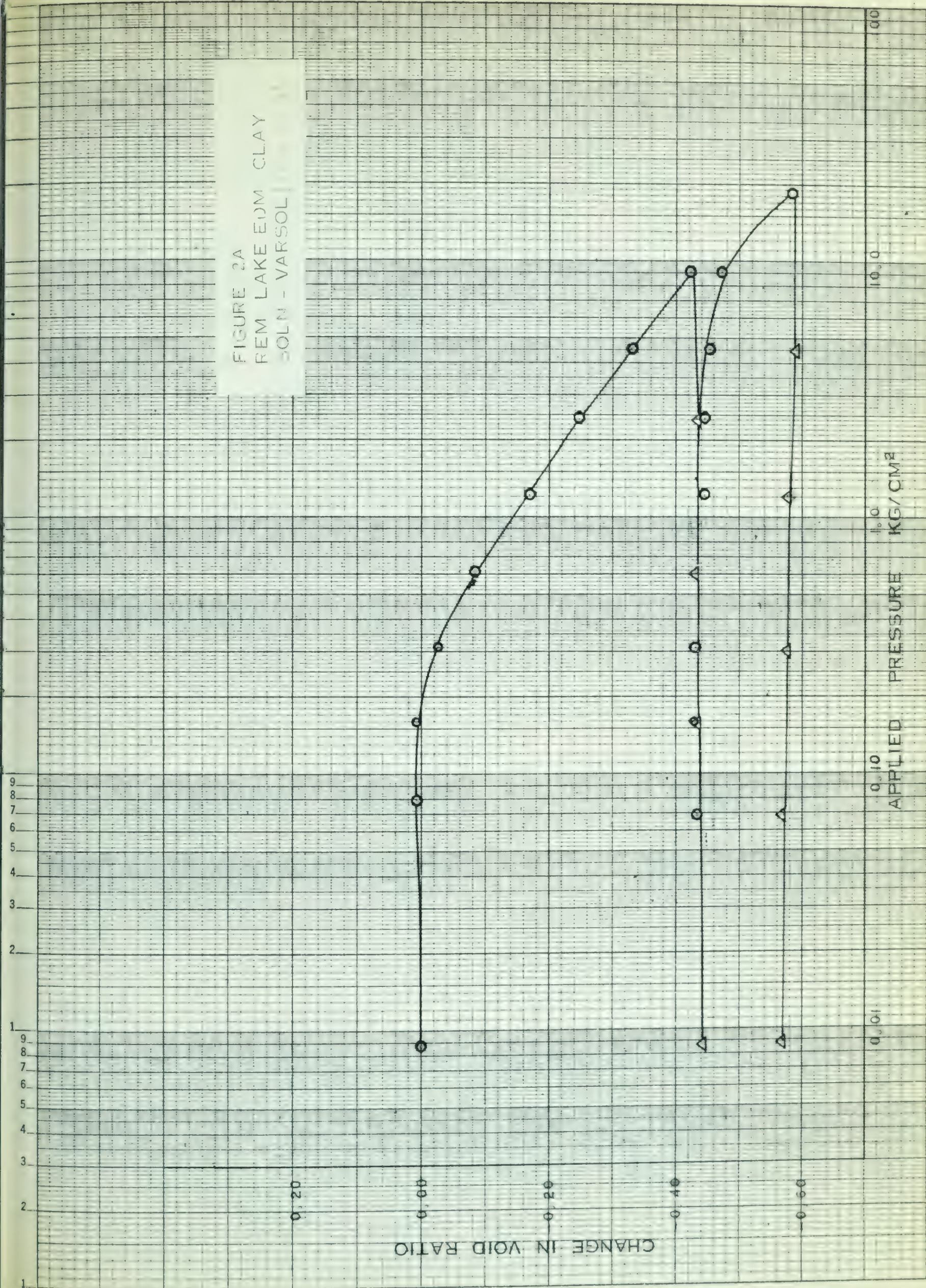
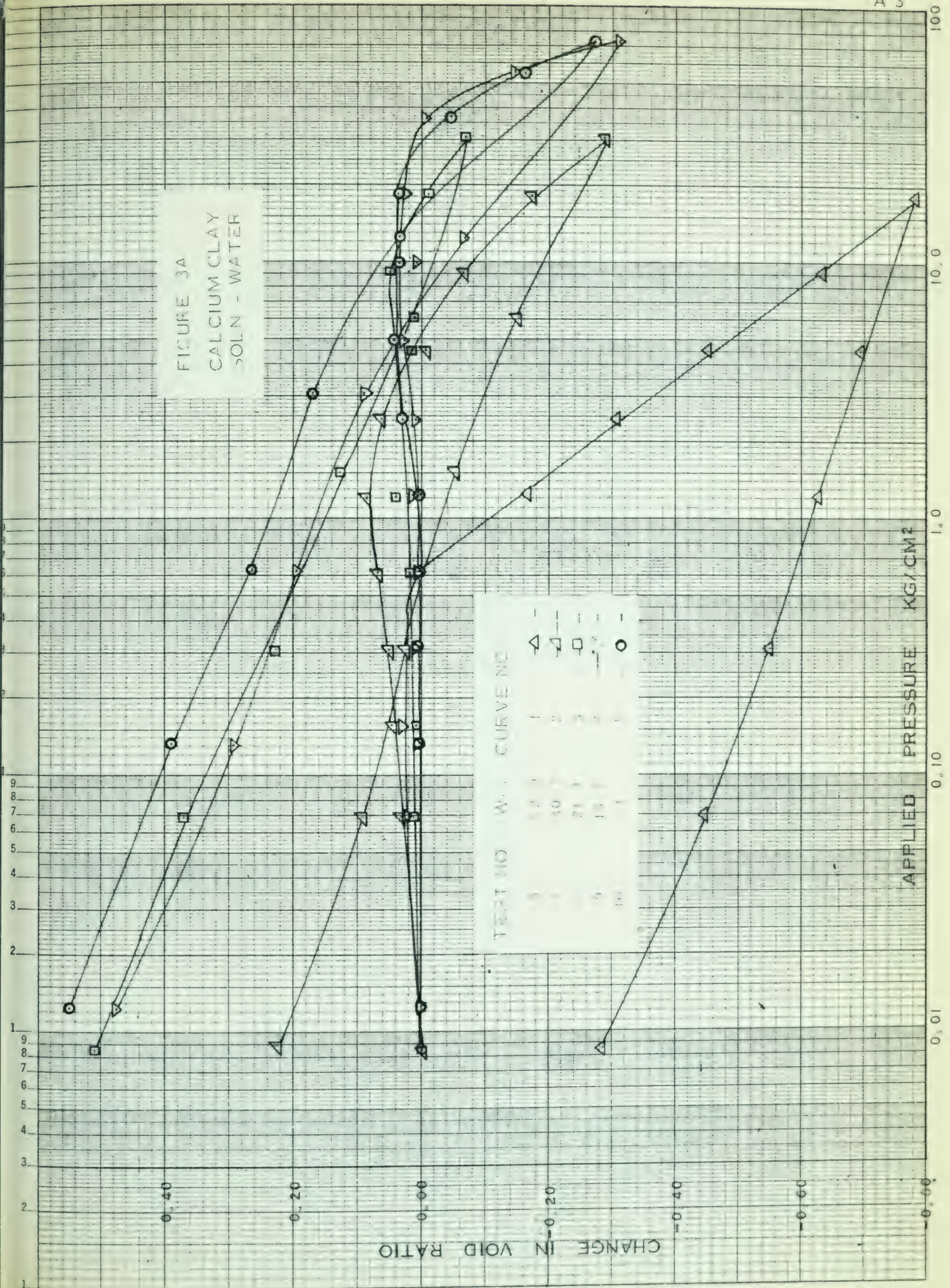


FIGURE 3A
CALCIUM CLAY
50LN - WATER



SOLN - 0.5 M C.A.

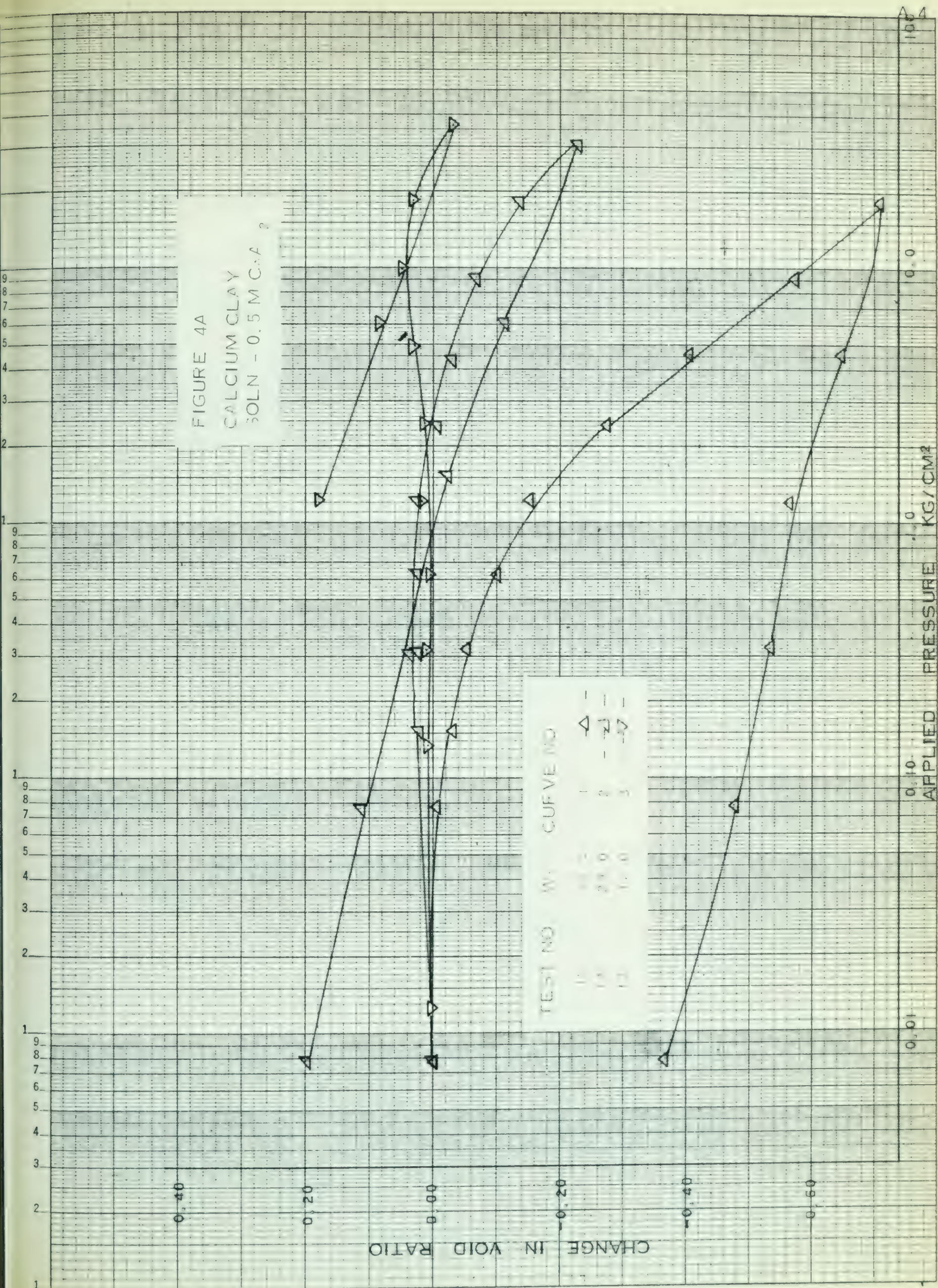


FIGURE 5A
CALCIUM CLAY
30LN 175 M C A 2

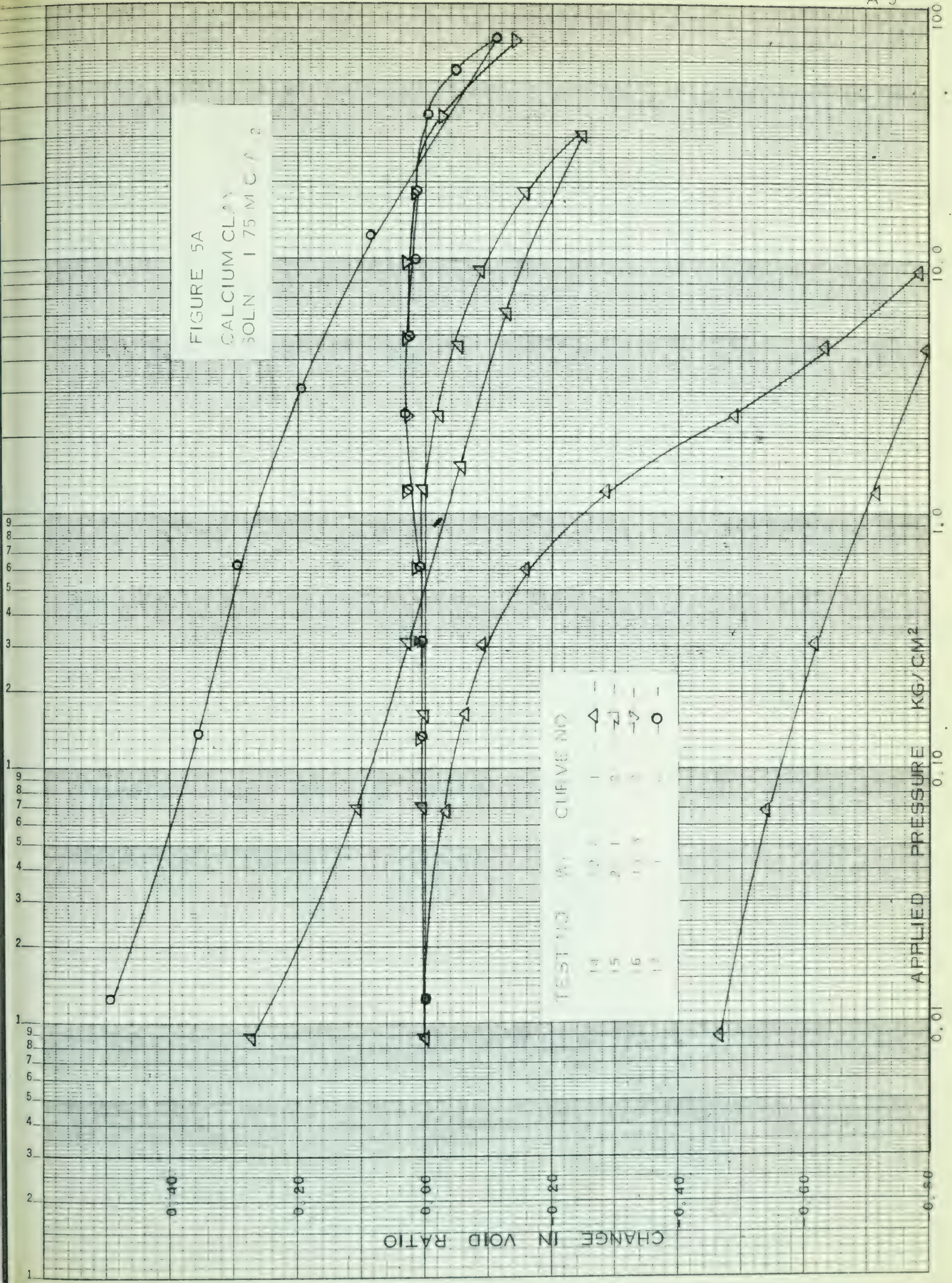


FIGURE 6A
SODIUM CLAY
SOLN - WATER

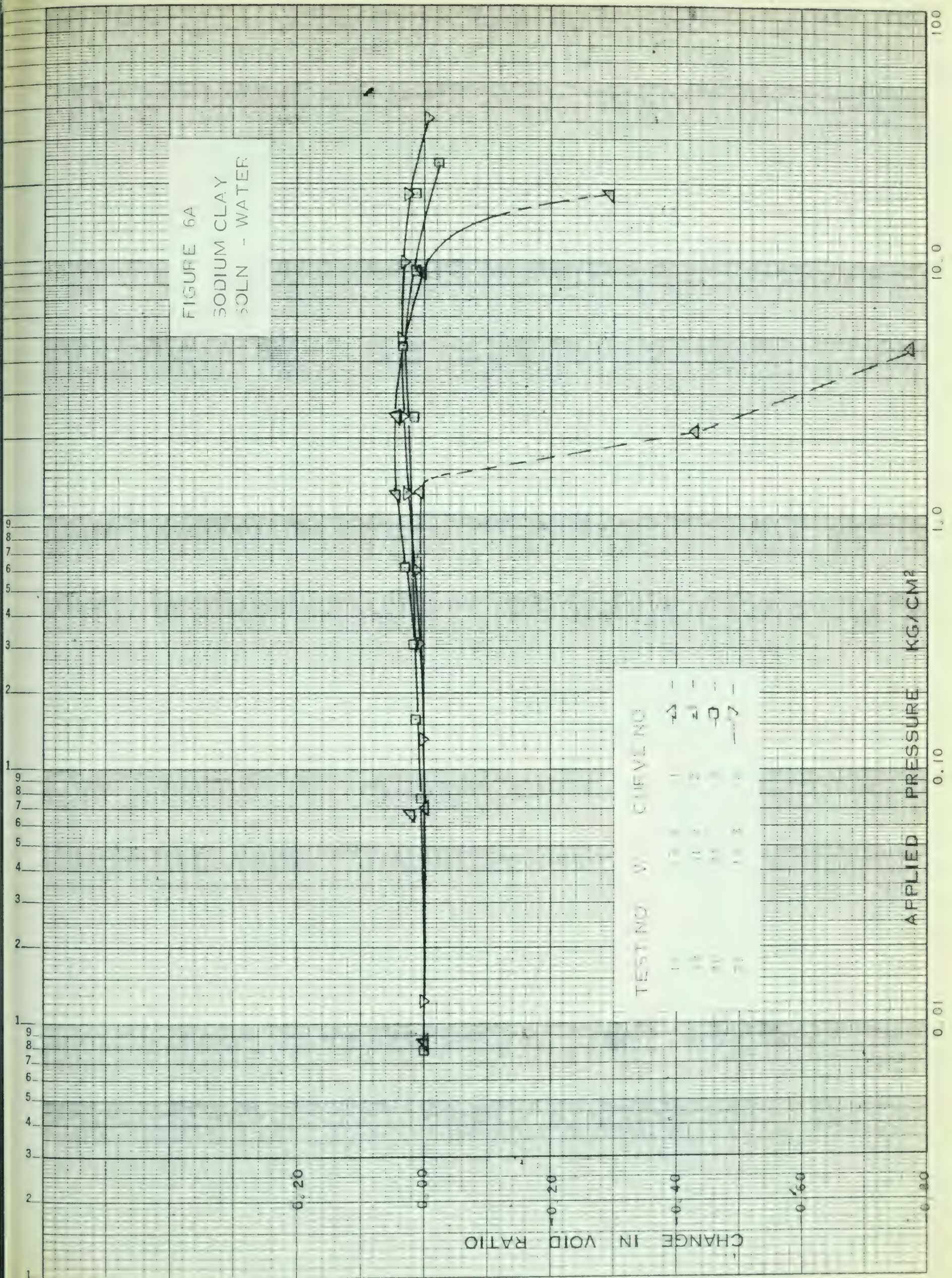


FIGURE 7A
SODIUM CLAY
SOLN. - 0.5 M N.A.

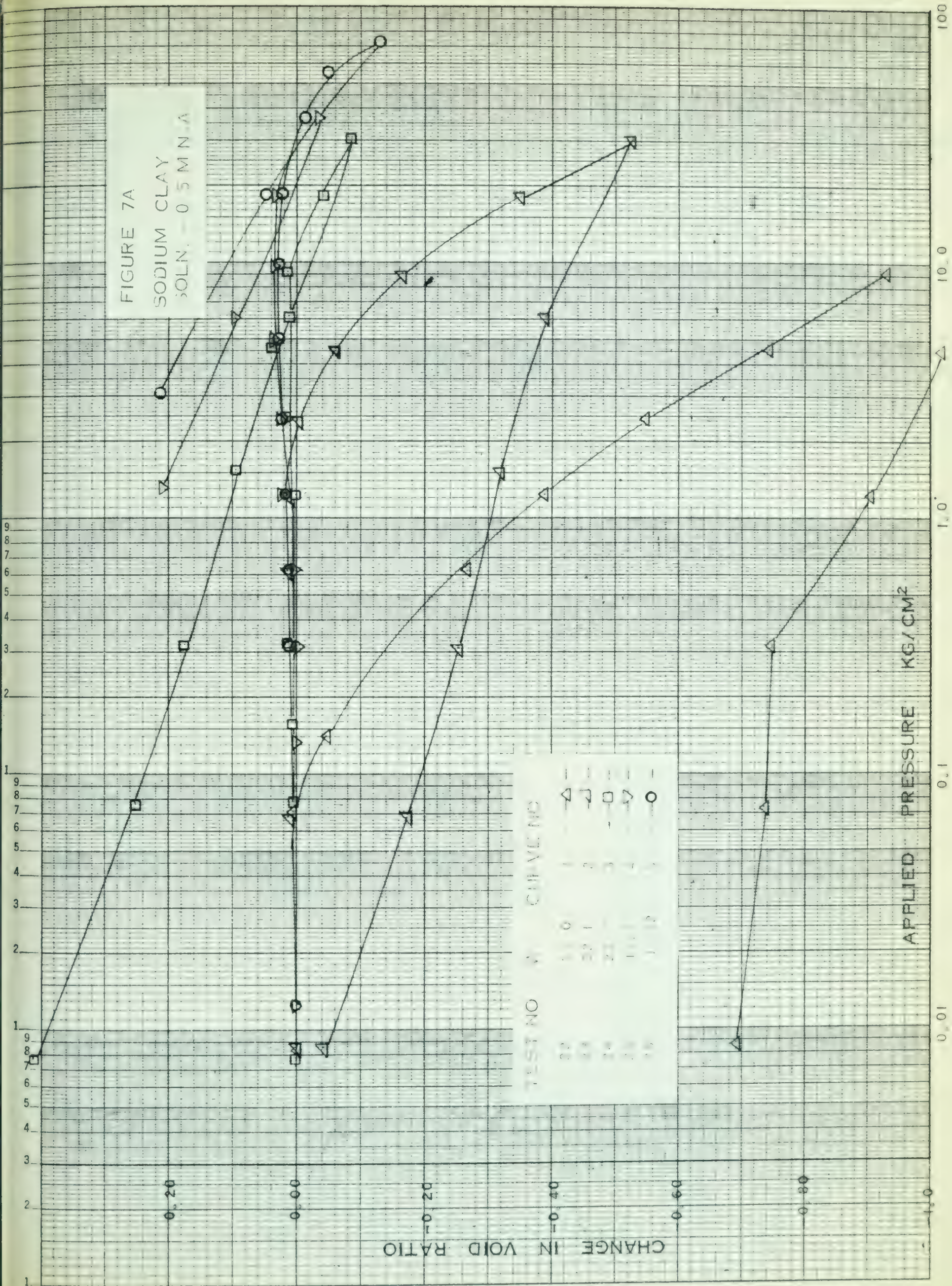


FIGURE 8A
SODIUM CLAY
50LN - 4 M N/A

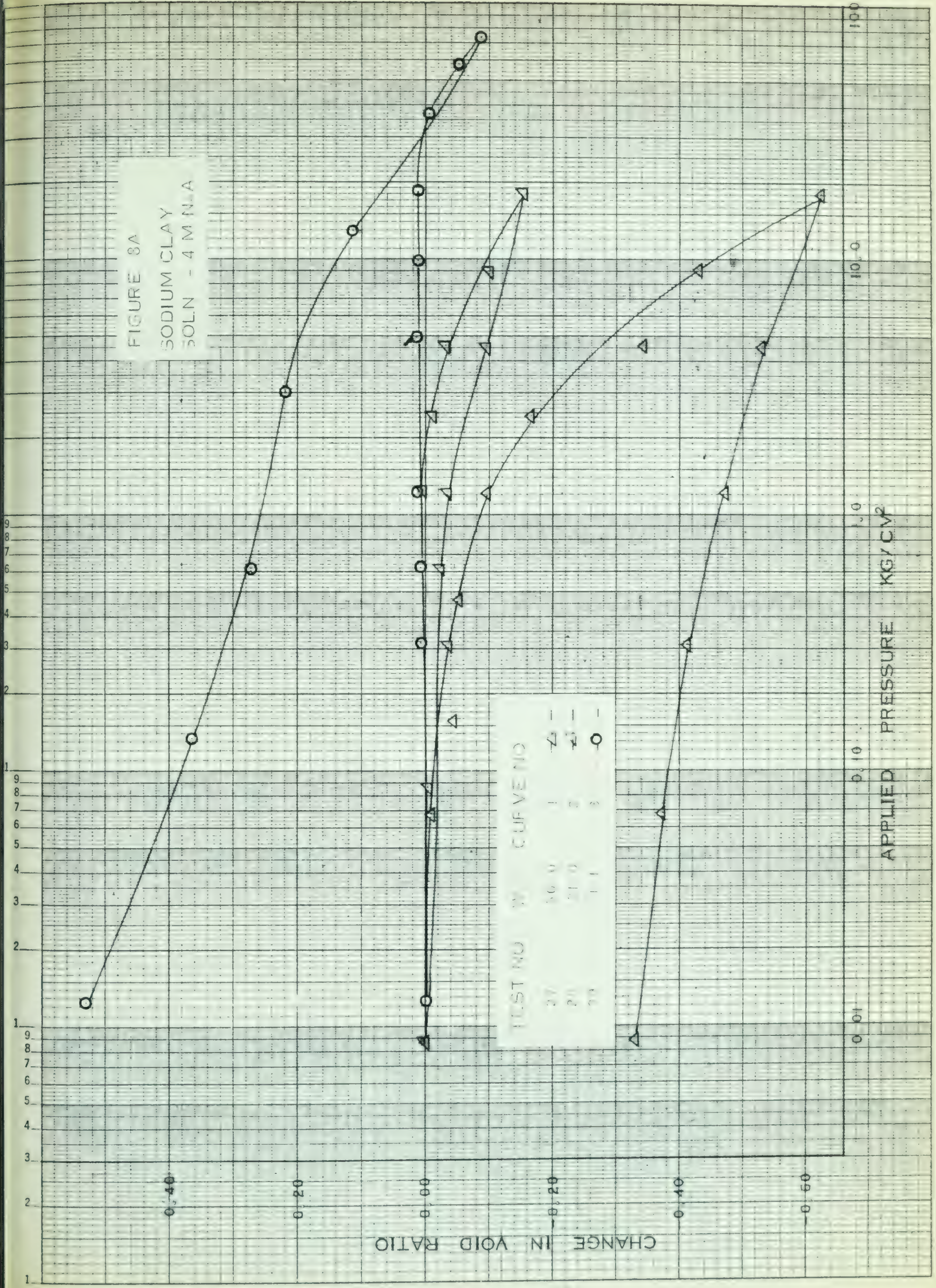


FIGURE 9A
 FULLY Saturated Ottawa Sand
 -OLN WATER-

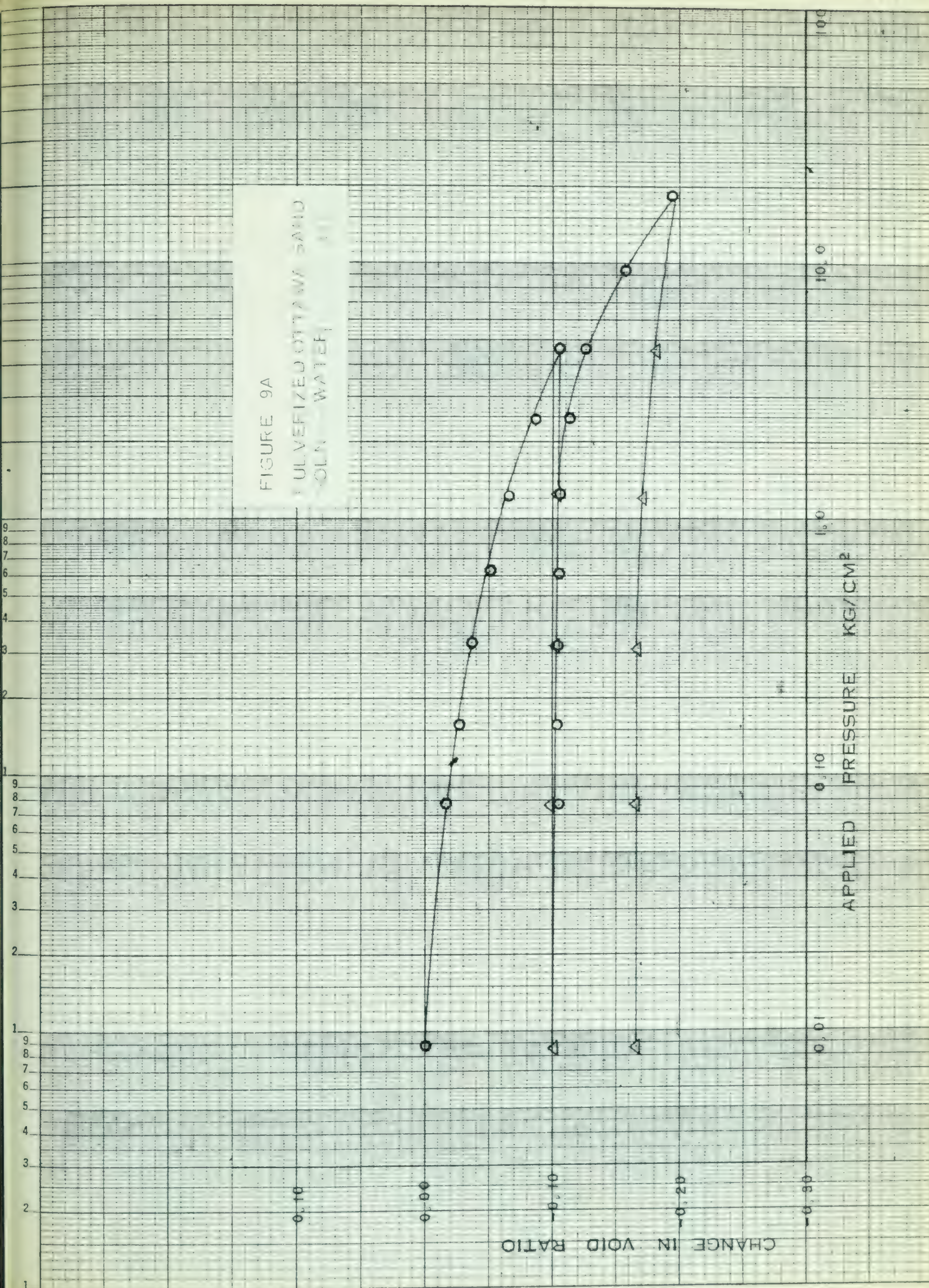
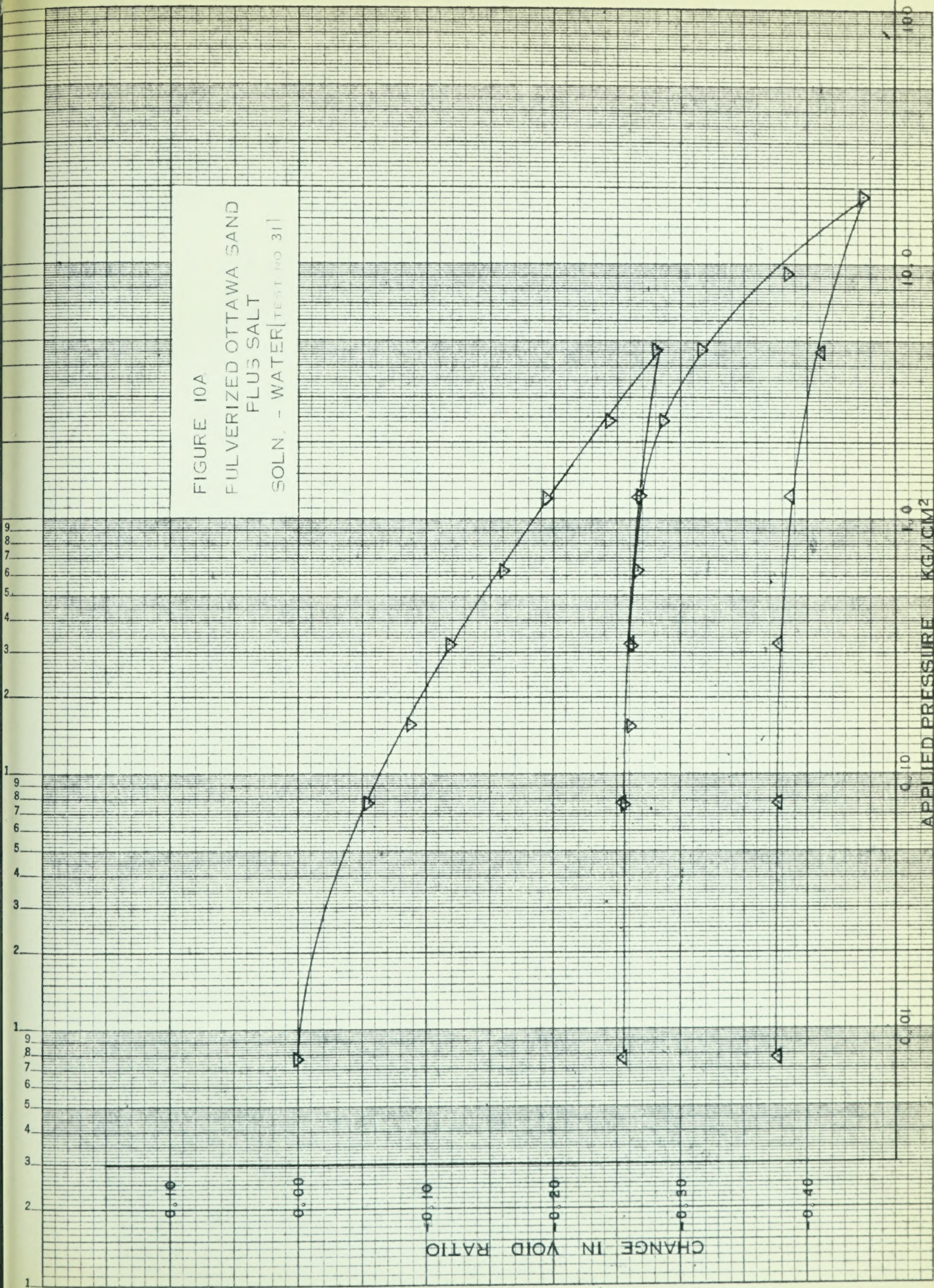


FIGURE 10A
PULVERIZED OTTAWA SAND
PLUS SALT
SOLN. - WATER [TEST NO 31]



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